JOURNAL
OF THE
AMERICAN WATER WORKS ASSOCIATION

VOL. 27 NOVEMBER, 1935 No. 11

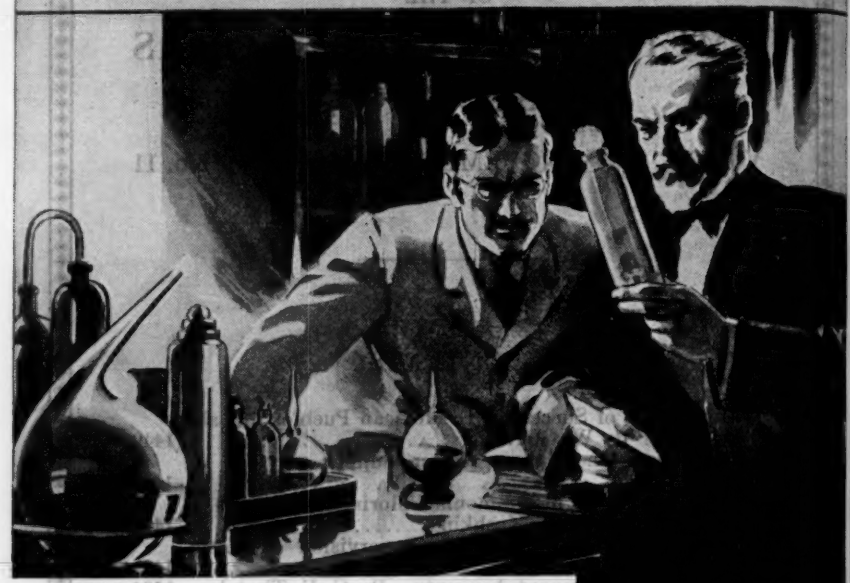
CONTENTS

Water Rights of Successors to Mexican Pueblos in California. By W. G. Irving 1449
Operation of the Athens Water Supply. By R. W. Gausmann 1458
Ammonia Dosage in Ammonia-Chlorine Treatment of Water. By M. L. Koshkin 1477
Accomplishments in the State Supervision of Dams. By George W. Hawley 1492
Handling of Delinquent Accounts. By C. E. Thatcher. 1504
The Lives of Water Works Structures. By G. A. Elliott. . 1509
Experience with Fluorine in Water. By Jane H. Rider . . . 1516
Effect of Meter Modernization in Germany. By H. W. Niemeyer 1525
Municipal Ground Water Supplies of Ontario. By W. B. Redfern 1533
Filter Washing Experiments at the Chicago Experimental Filtration Plant. By Herbert E. Hudson, Jr. 1547
Pipe Cleaning. By Charles Haydock 1566
Coagulating the Water of the Delaware River at Easton, Pennsylvania. By R. W. Haywood, Jr. 1572
Report of Committee on Electrolysis and Electrical Interference 1581
Report of the Committee on District Meetings of the Indiana Section 1589
Typhoid Fever in the Large Cities of the United States in 1934 1593
Standard Specifications for Riveted Steel Elevated Tanks and Standpipes 1606
List of Section Papers. Southwest 1626
Abstracts 1628

All correspondence relating to the publication of papers should be addressed to the editor, Abel Wolman, 2411 North Charles Street, Baltimore, Maryland.

Because of the necessity for rigid economy, no reprints of articles will be furnished to contributors free of charge. Reprints may be purchased at the usual prices.

• • FAMOUS VICTIMS OF WATER BORNE DISEASES



LOST TO THE CAUSE OF HUMANITY —LOUIS THUILLIER, SCIENTIST

STRANGE irony of fate that Pasteur, founder of modern preventative medicine, should lose not only his brilliant assistant, Thuillier, through death from cholera...but...in addition, two of his cherished daughters from typhoid fever—both water-borne diseases.

Chlorination of drinking water—the protection that even Pasteur's brilliant discoveries had not disclosed—is today available to all.

W & T Visible Vacuum chlorinators with their acknowledged dependability, with their low maintenance cost and simplicity of operation, make it possible to secure this prevention of water-borne disease transmission at a cost of less than one penny per person per year. That is an achievement of modern science and invention.

We urge you to think seriously of our slogan, "The Only Safe Water is a Sterilized Water."

Technical Publications 38, 157 and 158 describe W&T Visible Vacuum chlorinators in detail. A copy of one, or all, is yours for the asking.

WALLACE & TIERNAN CO., Inc.

Manufacturers of Chlorine and Ammonia Control Apparatus

NEWARK, NEW JERSEY Branches in Principal Cities...Main Factory, Belleville, N. J.

54-151

JOURNAL

OF THE

AMERICAN WATER WORKS ASSOCIATION

The Association is not responsible, as a body, for the facts and opinions advanced in any of the papers or discussions published in its proceedings. *Discussion of all papers is invited.*

Vol. 27 NOVEMBER, 1935 No. 11

WATER RIGHTS OF SUCCESSORS TO MEXICAN PUEBLOS IN CALIFORNIA

By W. G. Irving

(*Attorney, Riverside, Calif.*)

Four years ago I had the privilege of addressing you on some of the legal principles which define water rights in California. As you may remember, it was then stated that by reason of the adoption of the common law of England as the rule of decision in the courts of this state, insofar as the same was not repugnant to or inconsistent with the constitution of the United States, or the constitution or laws of this state, the doctrine of riparian rights became a part of our jurisprudence. A very unfortunate thing, as the history of litigation in this state will disclose. That by reason of the needs of the miners of '49 and subsequent years, and the growth of agriculture, viticulture and horticulture, the law permitting the acquisition of water rights by appropriation was developed by judicial decision. That as surface streams became inadequate to supply increasing demands underground sources were tapped, which led to the definition of the rights of the owners of land overlying such sources. These rights were discussed at your meeting in October, 1930. A fourth right was mentioned at that time, concerning which little was said. That is, the rights of municipalities in California who are the successors of Spanish or Mexican Pueblos. Fortunately for this state, there are few California cities which are able to assert and establish claims to

the use of water by citing the special ordinances of the Kings of Spain or the laws of the Republic of Mexico. Otherwise, the law of waters of California would, under the decisions of our courts, largely have been modeled on rules promulgated for the purpose of feeding the troops of Spain in America and furthering the policies and purposes of the monarch of that country. Anachronistic as the rules are which are invoked to measure the water rights of the successors of the Mexican Pueblos, nevertheless they are still given their full force and effect. It may interest you to know, in a very general way, what the rights of pueblos, or their successors, are in California today.

In 1860 the Supreme Court of this state said, in defining the rights of the City of San Francisco (as the successor of a pueblo) to lands therein situate, that for the purpose of the settlement of the new world,

> "It appears from the history of Spanish jurisprudence, that special attention was given in very early times to the establishment of cities, towns (then called "pueblos") and villages, and that particular laws were enacted for their foundation."

These laws permitted pueblos to occupy and possess about four square leagues of land, in trust for the inhabitants of the pueblo, with certain rights of control and alienation. With such rights we are not here concerned. As these pueblos were situate largely in arid regions, a permanent supply of water for irrigation and other purposes was essential to their perpetuation. It was therefore prescribed that pueblos should have rights in and to the flowing waters within their limits, and which waters were to be held in trust to be distributed to the lands of the pueblos and the lands of settlers within their limits, for irrigation, municipal and other purposes. This trust, however, was always subject to the supervision and control of the political authority which created the same. This right to take and distribute water of the pueblos was, by the Spanish and Mexican law, held to be a community right and as such was paramount and superior to the claims of individuals to the same waters. The right thus given to pueblos, to use the waters of streams passing therethrough, was both necessary, reasonable and just. Without such right no permanent settlements would have been undertaken; no pueblos would have survived; no development of the country could have been had. Certain limitations were placed upon the amount of water which could be taken and used by these early towns. These

were that the amount of water taken should be measured by needs, and the place of use should be within the confines of the pueblo.

Such is a very thin and sketchy description of the water rights of pueblos at the time California passed under the political dominion of the United States.

It was provided in the Treaty of Guadalupe Hidalgo, entered into between the United States and Mexico in 1848, that property owned and possessed by Mexican nationals in California should be unimpaired by the passing of California to the United States. It has therefore been held by our courts that a municipality founded upon one of these old Spanish-Mexican grants, having proven its claim, succeeded to all the rights, both to land and water, that were formerly possessed by the pueblo.

Shortly thereafter the leisurely and in many aspects commendable life of the Spanish-Mexican settlements was subjected to a gusty pioneering atmosphere. The discovery of gold in '49 extended an invitation to the adventurers of the world to come to California, and was accepted by many. These men, possessing the pioneering spirit, invaded the land—an invasion which still continues. As time passed and the mines gradually diminished in production, eyes were turned to soils fertile in character—needing but the application of water to develop their potentialities. Naturally water was taken where it could be found for the irrigation of these lands. Little regard was had or was known by these early American settlers of the rights of Mexican pueblos. They were not left long in doubt, however. Americans had also moved into and taken charge of the pueblos, and as the inhabitants increased in number and additional water was required, and as the supply was being diminished by demands thereon from others than those possessing pueblo rights, the rights of pueblos were investigated. Then began a series of appeals to the courts for a definition of those rights, which has not yet ended.

Shortly after the year 1848, the Supreme Court of California had occasion to pass upon some phases of the nature and extent of these rights. The first cases, however, involved only the title of cities to pueblo lands.

In 1879 a decision was rendered, in which the water rights of a pueblo were put in question. A riparian owner, upstream from a pueblo, claimed the right to use the waters of the common stream. The court held that the upper riparian owner was entitled to take water for his riparian lands, as against the other party, who was the

successor of a pueblo. In that case pueblo rights do not appear to have been stressed and were not discussed. The case was decided in accordance with the common law.

Two years later another action was decided, in which an appropriator from a river, which flowed through a pueblo, sought to enjoin the pueblo from closing his headgates and thereby depriving him of the water he asserted he had a right to use. The decision was in favor of the pueblo, for the reason that it had, for one hundred years, claimed all the water flowing in the river, that the appropriator knew of this claim and had recognized the same, had constructed his ditch and headgates by the permission of the pueblo; and therefore, as the pueblo needed the water, he was denied relief. There was no discussion in this case of the water rights of pueblos. It was held, however, that a pueblo could not sell water for use outside of its limits.

Again, in 1886, the rights of pueblos were discussed academically by the Supreme Court. It was then stated that the amount of water which a pueblo at any time may take from a stream is limited to the quantity necessary to supply its needs and those of its inhabitants. The decisions mentioned are generally in line with the general water law of california as we now know it.

In 1895, however, so far as I am aware, we find the first clear line of demarcation between the laws defining water rights generally and those defining the rights of successors to pueblos. The plaintiff in a case then decided was the owner of lands riparian to the upper reaches of a river and sought to divert water therefrom for the irrigation of its riparian land. A pueblo below sought to enjoin such diversion. Our court held that as against a pueblo even an owner of riparian land situate upstream from the pueblo, could not take or use the waters of the stream, if in so doing he reduced the flow of the same below that required to supply the needs of the pueblo. This decision is contrary to the general water law of California. Under the general law, a riperian owner can always exercise his riparian right as against users thereof below him, be they municipalities or individuals. Not so, it appears, as against pueblos. They, by reason of their Spanish or Mexican origin, are a favored class in our jurisprudence.

Four years later the rights of pueblos were further defined. It will be remembered that it was thought by laymen that these rights were in and to the surface flow of the stream. In 1899, however, it was held by the Supreme Court that water percolating through lands held in private ownership, which eventually drained into a stream in

which the pueblo possessed rights, was the property of the pueblo and that the private owner had no right thereto. Furthermore, at this time it was also declared that prior use does not measure the amount of water which a pueblo is entitled to take; that—"this right is measured by necessity, and if the needs increase in the future, the right will expand to include all that the needs require." Another departure from the general law of California to the advantage of pueblos. Under the general law, the owner of land overlying percolating water has the paramount right to the use thereof to the extent of his needs. Use by him of this water does not create the right and disuse will not destroy it. As against municipalities created in accordance with the laws of this state, his right will be protected to the same extent as against private invaders. But not so as to pueblos.

Again, at the time now referred to, the water right of a municipality was measured by the same yardstick as the right of a private individual. As against private owners a municipality was entitled to only such quantity of water as it had actually taken and put to beneficial use. Not so pueblos. While they can only take sufficient for the needs to be supplied at any particular time—yet, as their needs increase, their rights expand.

It may be that you are of the opinion that at least rights of pueblos are limited to the taking of sufficient water necessary to supply the lands within their original limits; in other words, to the customary four square leagues. If you hold such an opinion, you are wrong. In the year 1889 it was definitely settled that the right of the successor of a pueblo was not limited to the delivery of water to the lands and inhabitants of the original four square leagues, but expanded *pari passu* with the expansion of the area of the city, if and when it annexed additional territory. Thus the right of a pueblo's successor to use the waters of a stream flowing through the same, might be enlarged to the amount necessary to serve not only the original four square leagues, but to cover twenty square leagues or one hundred square leagues, or the whole of Southern California if it had power to annex that much territory. So far as I am aware, no municipality in California, other than the successor of a pueblo, possesses such expanding water rights.

SUMMARY OF RIGHTS

Such are the water rights of the successors of pueblos in California, granted by the ordinances of Spain and the laws of Mexico, as the

same have been interpreted by our Supreme Court. They may be summarized as follows:

(1) The right of a pueblo to use the waters of a stream upon which it is situate is, to the extent of its needs, paramount to the rights of riparian owners, and appropriators.

(2) If the stream is fed by percolating water, then the rights of the pueblo are paramount to the rights of owners of land overlying such waters.

(3) If the pueblo needs all the water, then riparian owners, appropriators and owners of overlying lands may look elsewhere for a place to live.

(4) The right of a pueblo expands with the growth of its population.

(5) The right of a pueblo expands as its territorial limits expand.

The only limitation on its rights seems to be that it cannot sell water outside of its limits.

STATUTE OF LIMITATIONS

Furthermore, this right possessed by the successor of a pueblo seems to be endowed with eternal life. It can be argued from some of our decisions, that there is no method known by which this right may be lost, impaired or abandoned. Men may die, principalities may rise and fall, change and decay may appear in all things—but pueblo rights, as they existed in 1848, march on forever.

That this is so, would appear from the latest pronouncement on such matter, made in 1930. The case then considered was the right of a successor of a Mexican pueblo established in 1834 to use the waters of a river which flowed through the same. It appears that little, if any, use had been made of these waters by the pueblo or its successor. In fact, the successor of this pueblo sought elsewhere for its supply. In the meantime several thousands of acres of fertile lands were brought into production by means of the use of the waters of the stream in question; cities and towns were established which depended for their existence on the continued cropping of the lands irrigated by these waters; water companies and irrigation districts were organized to divert such water; and for years this development continued. Recently the successor of this pueblo brought an action to have its right to the use of the waters of this river, to the extent of its needs, declared paramount to those of all parties using the same. To this action the defendants pled various defenses, one of which was the bar of the Statute of Limitations. In other words, they

alleged that they had used this water adversely to the pueblo under a claim of right for more than five years, and therefore the pueblo was barred from asserting any right thereto. To this defense the court said "No!" To quote the court:

> "Finally, and with direct reference to the defense of prescription, it may be stated as a general rule that no invasion of the rights of property which are held by a public or municipal corporation in perpetual trust for public uses can be held sufficient to furnish the basis of a defense based solely upon prescription. The cases fully supporting this general principle have already been cited, but will be further referred to in considering the subjects of laches and estoppel, to which we shall now address ourselves."

It would thus appear that the Statute of Limitations will not run against the successor of a pueblo.

LACHES

Another defense of the defendants was laches. Laches may be generally defined as "such negligence or omission to assert a right as, taken in conjunction with the lapse of time more or less great and other circumstances causing prejudice to the adverse party, operates as a bar in a court of equity." In the case we are now considering, years had elapsed since the defendants began their development; homes had been built and grown old; cities had been established and increased in population; and during that period this successor of a pueblo neglected to assert its claimed rights, with full knowledge of this development. To this defense the court again said "No." It, in effect, held that, as the waters of the stream were held by the pueblo, and its successor, in trust for its inhabitants, therefore, notwithstanding the fact that these waters had never been used by them, their rights could not be lost nor the public trust as to their administration and exercise be destroyed, either by adverse possession or by laches, or other negligence on the part of the officers of the municipality.

ESTOPPEL

Still another defense was interposed by the defendants in that case. It was the defense of estoppel. "An estoppel has been defined to be a bar by which a man is precluded from alleging or denying a fact in consequence of his own previous action, which has led another to so conduct himself that, if the truth were established, that other would be damaged."

To this defense the court said "No," for two reasons:—first, that the facts proven would not support the plea; and second (to quote the court):

"Even, however, if it were to be conceded that a right based upon estoppel could arise by virtue of mere acquiescence in its assertion as between private persons, we are satisfied that no such claim of right could come into being as against a municipal corporation, founded upon its mere acquiescence or that of its officials in the diversion by any number of upper appropriators, or even of upper riparian owners of the waters of a stream, to the use of the waters of which such public or municipal corporation was entitled *as a portion of its public rights and properties held in perpetual trust for public use.*"

As the court found that the facts tending to prove an estoppel were not sufficient for the purpose, there may be some hope that such a defense properly proven might be sustained as against a successor to a pueblo. The whole tenor of the decision discussed, however, would seem to be to the contrary.

EXERCISE OF GOVERNMENTAL FUNCTION

All of the defenses mentioned, when proven, are available against all municipalities of this state, except those who are fortunate enough to possess water rights as the successors of pueblos. The reason for the exception made in favor of pueblos is found in the character in which pueblos are said to possess and exercise their water rights. With reference to this matter it is said by our court that pueblo rights, and hence the rights of its successor, to whatever of the waters of a stream were from time to time required for the needs of the pueblo and of its inhabitants, were rights which were essentially governmental in character, as much so in fact as the rights of the ancient pueblo and modern city to the public square or streets within the same. It is axiomatic that the defenses of the Statute of Limitations, laches, estoppel and the like, are not available against the state or its agencies when acting in a governmental capacity, unless made so by statute. By declaring that a pueblo owns and exercises its rights to water in a governmental capacity, the court obviously makes the defenses mentioned of no effect as to it. Yet it is almost uniformly held by the courts of this state, and of the other states of the Union, that municipalities, organized under domestic laws and authorized to own and operate water rights, used for the purpose of supplying water for municipal purposes and for the use of their inhabitants, do so in a proprietary capacity; and as against such all of said defenses are

effective, if proven. In other words, the successor of a pueblo, performing the identical functions necessary to supply its inhabitants with water as are exercised by other municipalities, is exercising a governmental function and is immune from the ordinary laws regulating the water rights of other municipalities. If the successor of a pueblo seeks, obtains and distributes water from sources other than the stream which flows through its area, it does so in a proprietary capacity as to such water; but when it diverts water from the stream, its character immediately becomes governmental. One day the water delivered through its pipes may be delivered therefrom as the result of a governmental act; and the next day the act of delivering water through the same pipes, to the same consumers, for the same uses, is performed in a proprietary capacity. The act is the same in both cases, the purpose is the same, the use is the same. It is the source from which the water is obtained that appears to determine the character of the act. It is difficult, however, to appreciate the distinction in the character of the two acts. Nevertheless, such seems to be the law. When the ordinary municipality either seeks to protect or assert a right to water in California, it resorts to the general law of the state which defines water rights. When the successor of a Mexican pueblo seeks to protect or assert its rights to water—it ignores the ordinary rules on the subject that are found in the decisions. It is sufficient for it to rise and proclaim:—"Thus saith the King of Spain." The laws of California may be and have been modified, refined and in cases changed from time to time by our courts, as the public interest and welfare required, under the police power of the state. Not so the ordinances of the King of Spain and the law of Mexico, insofar as they pertain to the water rights of pueblos. They are still enforced in the form that they are thought to have been in, in the year 1848.

This is one phase of the water laws of California.

It may be, however, that there is now a possibility of modifying these ancient rights in the interest of the people of the state, without depriving any inhabitant of the successor of any pueblo of necessary water. It is not proper, however, for me to suggest how this change may be brought about, or what the action of the courts will be with reference thereto.

In so far as the law has been stated by the courts at this time, the rights of the successors of ancient pueblos are as described.

(*Presented before the California Section meeting, October 26, 1934.*)

OPERATION OF THE ATHENS WATER SUPPLY

By R. W. Gausmann

(General Manager, Water Works, Athens, Greece)

It is unusual if not unique for a population of some 800,000 inhabitants to become possessed suddenly of a complete new water supply. The operation of such a system presented many new problems, which would not have been problems in a city where the growth of the water supply had kept pace with the growth of the city. Not the least of these problems was the training of the personnel and one of the greatest aids in this training was and still is the Journal of the American Water Works Association.

It is partly in appreciation of the assistance afforded by this Journal, that this paper has been prepared. However, this article will only be a description of the operation of a water supply in a foreign country, as we do not consider that we have progressed sufficiently to present new methods or discuss new theories. It is hoped that these few notes may be of some interest to water works operators, because they describe the modern water supply of one of the best known cities in history and because conditions are quite different from those existing in the United States.

Possibly the fact which will appear most striking and be of the greatest interest to American water works operators, is that a modern flourishing Capital and Metropolis is now supplied with only 14 gallons of water per person per day and also that this very small quantity is more than five times greater than the supply of only a few years ago.

Other points which may be of interest are: the use of Continental standard pipe which is considerably lighter than the American standard, the small amount of unaccounted for water, experience with house connections of flexible copper tubing and the unusual growth of aquatic plants in the storage reservoir.

HISTORICAL

Only a century ago, a small Turkish village of scarcely 5,000 souls was all that remained of the once glorious city which has had more

influence on art, philosophy and government than any other city in the world. At that time the aqueduct,[1] started by the Roman Emperor Hadrian had been lost for centuries and it was not until 1847 that this famous structure was again put into use.

The city grew very rapidly after the Greeks had secured their independence and even before the World War, the water supply, which in a dry summer was only slightly over two million gallons per day, proved entirely inadequate. The influx of 300,000 refugees into the areas surrounding Athens and Piraeus in 1922, after the Asia Minor campaign, greatly aggravated this situation and in 1925 an agency contract was let with an American firm for the design and construction of a complete new water system and for its finance and operation in association with one of the principal Greek banks.

Work was started at once and in June, 1931 water was brought into the cities with appropriate ceremonies. By the end of that year all of the work was completed by the construction force and the entire system was turned over to the Operating Company.[2]

DESCRIPTION OF WORKS

The works consist of:

1. A collection and storage reservoir located about 20 miles from Athens with a capacity of 10.8 billion gallons and at such an elevation that all but 12 percent of the water is distributed by gravity.

2. An aqueduct for conveying the water to the city which in turn consists of: a tunnel 8⅓ miles long, 8.15 feet high and 7.65 feet wide, with a capacity of 350 c.f.s.; two small tunnels having a combined length of 7,830 feet, height 5.25 feet and width 4.6 feet; and a cast iron pipe siphon 18,910 feet long and about 36 inches in diameter with a carrying capacity of 18.5 m.g.d.

3. A purification plant having a capacity of 15 m.g.d., consisting of: (a) A raw water storage in the tunnel of 8½ million gallons. (b) A spray type aerator having a capacity of 18½ million gallons. (c) A retention basin of 2 million gallons capacity providing for a normal

[1] Major James F. Case, M. Am. Soc. C. E. dec., has published in the Transactions of the A. S. C. E. a very interesting article on this aqueduct.

[2] A description of the design and construction of this work may be found in an article entitled "The New Water Supply Works for Athens, Piraeus and Environs" by R. W. Gausmann, which is on file in the Engineering Societies library. A brief digest of this article was published in Civil Engineering, January, 1933.

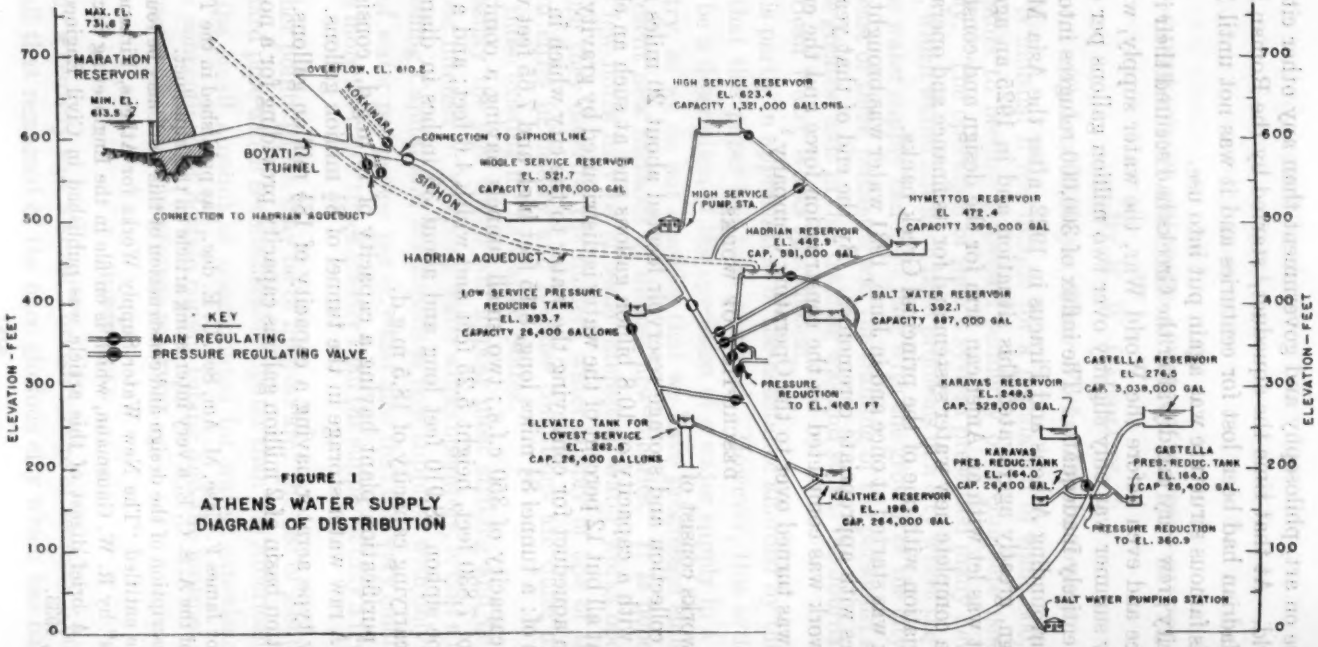

MAX. EL. 731.6
MARATHON RESERVOIR
MIN. EL. 613.5
OVERFLOW, EL. 610.2
KOKKINARA
BOYATI TUNNEL
CONNECTION TO SIPHON LINE
SIPHON
CONNECTION TO HADRIAN AQUEDUCT
HADRIAN AQUEDUCT
MIDDLE SERVICE RESERVOIR EL. 521.7 CAPACITY 10,870,000 GAL
HIGH SERVICE RESERVOIR EL. 623.4 CAPACITY 1,321,000 GALLONS
HIGH SERVICE PUMP. STA.
HADRIAN RESERVOIR EL. 442.9 CAP. 991,000 GAL
HYMETTOS RESERVOIR EL 472.4 CAPACITY 396,000 GAL
LOW SERVICE PRESSURE REDUCING TANK EL. 393.7 CAPACITY 26,400 GALLONS
SALT WATER RESERVOIR EL. 392.1 CAPACITY 687,000 GAL
PRESSURE REDUCTION TO EL. 410.1 FT
ELEVATED TANK FOR LOWEST SERVICE EL. 262.5 CAP. 26,400 GALLONS
KÁLITHEA RESERVOIR EL. 196.6 CAP. 264,000 GAL
KARAVAS RESERVOIR EL. 248.3 CAP. 528,000 GAL.
CASTELLA RESERVOIR EL. 276.5 CAP. 3,038,000 GAL
KARAVAS PRES. REDUC. TANK EL. 164.0 CAP. 26,400 GAL.
CASTELLA PRES. REDUC. TANK EL. 164.0 CAP. 26,400 GAL.
PRESSURE REDUCTION TO EL. 360.9
SALT WATER PUMPING STATION
KEY
MAIN REGULATING
PRESSURE REGULATING VALVE
ELEVATION - FEET
0
100
200
300
400
500
600
700

FIGURE 1
ATHENS WATER SUPPLY
DIAGRAM OF DISTRIBUTION

FIG. 1

retention period of 3 hours. (d) 10 filter units each rated at 1,500,000 gallons daily and having an area of 525 square feet. (e) A clear water and distribution reservoir of nearly 11 million gallons capacity.

4. A distribution system 20 square miles in area serving 800,000 people. At the end of 1934 the system contained a total of 600 miles of pipe line. Fifty-nine percent is approximately 3-inch (80 mm.) in diameter which is the smallest size pipe used except for about 4 miles of pipes annexed to the new system. Twenty two percent is about 4-inch (100 mm.). The largest pipe is 36-inch but there is only a short length. There are 6,650 valves i.e. approximately 11 per mile, of which nearly 75 percent are 3-inch. The largest size is about 30-inch (750 mm.). There are seven distributing reservoirs having a total capacity of nearly 16 million gallons, about 1½ days supply. This includes the clear water reservoir at the purification plant. In order to keep pressures within normal limits, the city has been divided into four major zones according to elevation. In general the zones are from 100 to 200 feet in height. Small pressure reducing reservoirs and pressure reducing valves help to further reduce pressures so that the normal maximum is not much over 50 pounds per square inch. A little less than 12 percent of the total consumption has to be pumped to the highest parts of the city. A diagrammatic profile of the system is shown on figure 1.

5. House connexions which extend from the main to and including the meter which, in general, is set in the sidewalk, just outside the house. (There is no danger from frost.) At the end of 1934 there were nearly 78,000 house connexions.

QUANTITY AND CONSUMPTION

An American water works engineer undertaking work or even studying installations in the vicinity of the Mediterranean Sea, must revise his ideas concerning per capita consumption. For instance in 1931, 17 of the Mediterranean cities (not including Rome, which is liberally supplied with 100 gallons per capita per day) had an average supply of only 28 gallons per capita. This includes: Alexandreia, 38; Alger, 27; Beyrouth, 23; Genoa, 48; Marseilles, 35; Messina, 17; Nice, 36; Port-Said, 21; Trieste, 38 and Tunis 15.

Athens with a per capita consumption in 1934 of only 14 gallons per day, consumes only half the average of the other Mediterranean cities but this consumption is rapidly increasing. The 1934 consumption was 21 percent greater than that of 1933.

In comparing the water supply of Athens with American cities, the total number of inhabitants may serve as a basis for the commercial side of the business, but a comparison of the so-called technical features i.e. supply, purification, size of mains, etc. can only be satisfactorily made with a city having from $\frac{1}{10}$ to $\frac{1}{5}$ the number of inhabitants, i.e. cities having a population of from 80,000 to 160,000 people.

Estimates of the available yield of the Marathon Watershed based on the most meager data were made in 1925 before the works were started. These estimates fixed the probable yield at "500 lit/sec. or more" this equals about 14 gallons per capita.

A study made in 1932 based on seven years' measurement of actual run-off together with seven more years of rainfall data gives the calculated minimum yield as 526 lit/sec. and it is estimated that allowing for an increase of population and an increase in per capita consumption, the present sources will give an adequate supply, at least until 1946–49. It is planned to repeat this study at 5 year intervals. The new data which are continuously becoming available will naturally tend to increase the reliability and accuracy of such calculated predictions.

In the meantime a start has already been made on the collection of field data from more distant sources which may be used in the future. The total annual yield of all of these sources appears to be about 2½ times that of the Marathon watershed, but a part of this total yield will not be available as it will have to be used for local irrigation. In any event, if money is provided in time, there is no reason for Athens and its environs to suffer from a lack of water for many years to come.

QUALITY OF WATER

Samples are taken at frequent and regular intervals of the raw water in the reservoir, of water in various stages of treatment at the purification plant and of water as distributed at the house taps in various parts of the city. These samples are tested in the Company's laboratory. For the year 1934 the results were as follows:

The raw water from the reservoir is subjected each week to a physical, chemical and bacteriological examination.

Physical examination. Appearance—good 80 percent of the time. Color—none 70 percent of the time. Odor—none. Taste—none 90 percent of the time. Turbidity—2 to 120 p.p.m.—average 11.

Physically this raw water would be classified as good nearly all of the time.

Bacteriological examination. Number of bacteria per cc. on Agar 37°C.—24 hrs., 1 to 550, average 49. Number of B. Coli (Coli Aerogenes group) per liter 0 to 10,000, average 1,070. The high counts of bacteria noted above occurred after heavy rains. During the remainder of the time the count was low.

Chemical examination. Complete chemical tests are made weekly of both the raw and purified water. The raw water contains no free chlorine, has an average hardness of 141 p.p.m. of which only 13 p.p.m. is non-carbonate. Average pH is 7.5, no iron or manganese, average free carbon dioxide 3. Free ammonia 0.02 p.p.m. Chlorides 28 to 35, average 31 p.p.m.

The treatment at the plant consists of aeration, coagulation, sedimentation, filtration and chlorination. During the past year the amount of alum used per month varied from 17.26 p.p.m. in February to 2.50 p.p.m. in the Fall. Ammonia varied from 0.155 p.p.m. in February to 0.135 p.p.m. Free chlorine varied from 0.386 in February to 0.369 p.p.m. in March. Tests indicate that 72 percent of Al_2O_3 settles in the sedimentation basin and 28 percent is retained on the filters. Filters operated from 15 to 22½ hours per day and from 27 to 55 hours between washes. During the summer months when the wash water recovery plant was in full operation 1.20 percent of the total water filtered was used for washing and other purposes of which 0.93 percent was recovered leaving a net loss of 0.27 percent.

The treated or purified water is tested daily and the results of the tests are as follows:

Physical examination. The appearance is always good and it has no color, odor, taste or turbidity.

Bacteriological examination. The number of bacteria per cc. on Agar 37°C. after 24 hours varies from 0 to 6 and averages 1. There are no B. Coli.

Chemical examination. This is made only once a week. Because of mixing the water from the reservoir with the underground water from the tunnel and the Hadrian aqueduct supply the average hardness is increased to 174 p.p.m. of which an average of 16 is non-carbonate. The purified water having been treated with chloramine has an average of 0.197 p.p.m. of free chlorine and 0.086 p.p.m. of free ammonia. The pH content is 7.55. There is no iron or manganese and the average free carbon dioxide is 2.6 p.p.m.

General. From the above it is readily apparent that the water delivered to the customers, although hard, is excellent in quality.

While there has never been any odor to the water, activated carbon has been purchased and the plant has been equipped so that such an emergency could be readily handled.

In designing the plant, full provision was made for extending it whenever this necessity arises. It is now proposed that when this extension is made it might be advisable to construct a water softening plant. This would reduce the hardness to 80–85 p.p.m. and while a very small charge would have to be added to cover the cost of installation and operation of the softening plant, the savings made through the use of a softer water would more than cover the necessary charge.

Partly with a view to be more fully informed and also as a means of self protection, frequent examinations have been made of wells used for water supply in the vicinity and of fruit bought in the open market, in order to determine the possible sources of contamination. Almost invariably the wells have shown a high B. Coli content. The fruit purchased in the market and from the street vendor is invariably contaminated. Fortunately since the establishment of the water works there has been no epidemic of any kind and there has also been a reduction of deaths from typhoid from 44 per 100,000 in 1923 and 17.4 per 100,000 in 1931 to 14.2 per 100,000 in 1934.

It is not unreasonable to predict that in the future when the simple rules of sanitation are known and appreciated by all, that the death rate from typhoid and other similar water borne diseases will be a function of the efficiency of the staff.

STRUCTURES

Under this heading it is proposed to describe briefly how the various units and structures have functioned during the past 3 to 4 years of operation.

Dam, leakage. The marble dam at Marathon is probably unique as to materials, but it is a quite normal masonry dam, curved in plan and with a gravity section. Provision was made in the design to resist uplift, but at the same time the foundation was heavily grouted and drained. The total leakage is collected in a drain and is measured weekly. This leakage has been reduced gradually by natural means only, from 1.201 lit/sec. (equals 19 gal/min.) in 1929–30, to 0.686 lit/sec. (10.9 gal/min.) It is presumed that this reduction is

due to the gradual closing up of the small water passages with lime found in the water and also in the cement.

Expansion: To provide for normal expansion and contraction of the masonry, eleven contraction joints were placed at intervals of 82 feet throughout the length of the structure. These joints extend from the top of the dam to the original surface of the ground. Bronze pins have been set on either side of these joints and measurements have been made each month, with a micrometer calipers, of the distance from "out to out" of the pins. The total movement of the masonry as indicated by these measurements is 12.25 mm. or slightly more than one milimeter for each one of the eleven joints. The opening and closing of the joints coincide with the seasons.

Reservoir, supply. The dam was completed and the filling of the reservoir was started in 1929. During 1930 it filled to within 3 meters of the flow line and in January 1931 it overflowed for the first time. The first water was drawn in June of the same year. During the winters of 1932, 1934 and 1935 it overflowed and in 1933 it only lacked a few inches of going over the spillway. The maximum draw down, approximately 11.5 feet, occurred in November, 1934, and was equivalent to nearly 20 percent of the available capacity.

During the winter of 1934–1935 flash-boards were placed on the spillway for the first time with the possibility of increasing the available capacity by about 4 percent.

Evaporation: Measurements have been made of the evaporation since 1926. A floating pan was first used in 1929 and the new type of floating pan recommended by the U. S. Geological Survey (Report Special Comm. on Irrigation Hydraulics, Amer. Soc. C.E.) was placed in service in February, 1933 while the new type of land pan (U. S. Weather Bureau) was first used in July of the same year. The measurement of evaporation by reliable means has averaged a little over three feet per year or approximately $5\frac{1}{2}$ to $7\frac{1}{2}$ percent of the total capacity of the reservoir.

Measurements of the inflow and outflow of the reservoir have been used to check the volume of evaporation measured by the pans during the past year. The actual evaporation was 69 percent of the volume measured by the land pan. The special Committee on Irrigation Hydraulics of the Amer. Soc. C.E. recommended an average of 70 percent. The actual evaporation was 80 percent of the volume measured by the floating pan which is the same coefficient recommended by the above Committee.

Intensive runoff: The most intensive runoff occurred on November 22, 1934 and amounted to 345.5 m^3/sec. or 266 c.f.s. per square mile of drainage area, for 15 minutes. For one hour the rate was 288 m^3/sec. or 221 c.f.s. per square mile and for three hours 241 m^3/sec. or 185 c.f.s. per square mile. The spillway was designed to carry a flood of 520 m^3/sec. or 400 c.f.s. per square mile with a head of 6.5 feet or 727 m^3/sec. or 560 c.f.s. per square mile with a head of 8.2 feet, so that there would have been a very ample capacity to take care of the November, 1934, flood had the reservoir been full when it started.

Ordinary runoff: The monthly coefficient of runoff varies greatly, but in general the annual relation between rainfall and runoff approximates the curve prepared by the late C. E. Grunsky for California Conditions (See p. 223 Proceedings Amer. Soc. C.E. for 1921). The actual annual percentage varies from 8.5 to 31.7, the average being 19.9.

Plant growth: Eight distinct varieties of aquatic plants have made their appearance in the water of the reservoir. One variety attains a height of 30 to 35 feet. Up to the present there has been no taste or odor from any of the plants, but their appearance on the surface of the water is unsightly and for this reason a considerable amount of time and money is spent each summer in removing them. This past summer the growth was considerably less than the preceding summer and in turn the growth during that summer was less than two summers ago. It is too early to be sure of the reason for the reduction in growth, but it may possibly be due to the very complete removal which has been made before the plants have a chance to seed themselves. The maximum number of tons of weeds removed during any season is estimated at 1200 tons in 1932. During 1934 nearly 500 tons were removed. The expense of removing the weeds was about 50 to 75 cents per ton.

Temperature: The temperature of the water at various depths is measured at regular and frequent intervals. Examining the results of these measurements for the past 3½ years and considering only the monthly averages we find that the maximum surface temperature was 25.7°C. (78°F.) in August and the minimum 7°C. (45°F.) in February. At a depth of approximately 100 feet the temperature varied from a minimum of 6°C. (43°F.) in February to 12°C. (54°F.) in December. The intake chamber is so arranged that water can be drawn at practically any depth, but actually water is drawn from a depth of

50 feet in winter and 115 feet in summer. In general the temperature of the water drawn into the tunnel has never exceeded 12°C. (54°F.). The so called "turnover" has not occurred in this reservoir.

Gambusia: In 1928, 190 "Gambusia" or "surface minnows" were placed in the partially filled reservoir as one of the means of preventing malaria among the construction forces. The small fish have done their work well as there are practically no mosquitoes in the vicinity of the lake. They entirely disappear during the winter months but in the spring they reappear by millions. A study is being made of the life history of these creatures which are not true fish as they bear their young alive and do not lay eggs.

Sedimentation: Measurements have been made from time to time to determine if any sediment was being deposited in the reservoir. Up to the present the amount is so small that it presents no problem. It is all deposited where the rivers enter the reservoir.

Gate chamber. The so-called "gate chamber" houses the controlling devices of the entrance to the tunnel. The intake consists of two identical and independent parallel units each capable of conveying the maximum amount of water required, with arrangements so that water may be drawn from any level. An enclosed needle type valve is used to control the quantity of flow. The valve operated quietly and in general without any difficulty for about three years when it was found that a portion of the needle had become so deeply pitted that the casting was perforated and the valve would not close tightly. A new needle of slightly different design is now being constructed of bronze instead of cast iron. The seeming cause for this rapid pitting is the continued use of the valve at a relatively very small opening.

Tunnel. Three annual inspections of the 8-mile tunnel have shown it to be in perfect condition.

Siphon line. Despite strong recommendation to the contrary, it was ruled that this line be constructed of cast iron rather than steel pipe. The line has been in full service for about $3\frac{3}{4}$ years and during this time the cast iron pipe has burst four times. The exact cause has not been definitely determined, but it is thought that it may have been due to unequal settlement of the line, which unequal settlement may in turn have been due to slight leaks at the joints caused by expansion and contraction of the line. Fortunately up to the present time the breaks in the siphon line have been repaired in time so that there was no lack of water within the cities. However,

as the rate of consumption grows, the allowable time within which a break may be repaired without causing a deficiency in the supply, will become less and measures may have to be taken to ration the existing supplies in the service reservoirs.

In order to determine quickly how long the available supplies will last under varying consumptions a set of graphs has been prepared on transparent cloth. By superimposing the curve of inflow (which will be by-passed through the old Hadrian aqueduct), over a graph giving varying quantities of supply in the service reservoirs and having the time element as an ordinate and then over these two placing a third graph of varying consumption, it is possible to determine at what time the available supplies will be exhausted. At the same time an estimate would be made of the time required to repair the pipe. On the basis of these two estimates the necessity for and degree of rationing the available supply may be quickly and reasonably determined.

During construction, provision was made for the construction of a parallel line whenever required. It is expected that this parallel line will soon be constructed as it will be urgently needed to convey the maximum demand during the summer months. It will be of steel rather than cast iron pipe.

Distribution system, pipe lines. With the exception of the central main which is nearly 10 miles long and extends from the middle service reservoir to Piraeus and is steel pipe, all of the pipe in the distribution system is cast iron of European standard.

Size or pipe: As noted in the brief description of the distribution system, the percentage (81%) of small pipe (3- and 4-inch) is far greater than in an American city of the same size. This is due: (a) To the absence of any provision for fire protection as Athens is nearly a fire proof city and is partially protected by a separate salt water system and (b) to the very low per capita consumption already mentioned.

Weight: The Continental standards are considerably lighter than American standards. For instance the weight of the cast iron pipe in the distribution system as laid up to 1931 by the Construction Company, and having a length of 532 miles was 32,643 metric tons whereas had the same line been constructed with American standard pipe of the same theoretical strength it would have weighed 37,981 metric tons or 16+ percent more. In British standards it would have weighed 34,733 or 6 percent more. For simplicity the steel line

has been included in both of the above calculations using the equivalent weight of cast iron pipe of the same strength. Class "B" is the lightest American pipe considered.

As a further example of the difference between Continental and American pipe of the same theoretical strength, it may be noted that for the size nearest to 3-inch (80 mm.) the thickness of the Continental pipe is 9.0 mm. whereas the American is 10.67. For the 4-inch (100 mm.) size, the Continental pipe has a thickness of 9.0 mm. and the American, 11.43 mm. For the 6-inch (150 mm.) the thicknesses are respectively 10.0 mm. and 12.19 and for the 8-inch (200 mm.), 11.0 mm. and 12.95.

Breaks: During 1934 there were 44 breaks in the pipe lines of the distribution system. This is equivalent to 0.071 breaks per mile of line. 37 of the 44 breaks were in the 3- and 4-inch sizes. The total number of breaks per mile per year has been decreasing. Thus in 1933 there were 0.106 and in 1932, 0.147.

The record of breaks for 1934, i.e. 0.071, compares fairly well with American records of 0.013 to 0.052 when the size of pipe and the difference in weight is considered. Most records show more breaks in the smaller sized pipe. Records in England, where pipe sizes are only slightly heavier than Continental standards, show breakages of 0.126 to 0.412 per mile per annum.

Bursts of joints: All pipe joints are thoroughly calked with lead and jute and tested under a pressure of at least 50 percent more than the static pressure. Nevertheless joints burst. During 1934, there were 188 bursts of joints. This equals 0.314 per mile of pipe. The number of bursts has reduced from year to year in the same way as the breaks. In 1933 there were 0.375 per mile and in 1932, 0.500.

Electrolysis: Stray currents from the electric railway have caused the phenomenon known as electrolysis in two locations on the steel main. The pipe was pitted so deeply and over such a large area that to date 1840 feet of 30-inch main (750 mm.) and 1000 feet of 24-inch main (600 mm.) have had to be taken out, repaired and replaced.

In repairing the pipe, the coating was entirely removed, the pipe surface thoroughly cleaned and then the pitting was refilled by oxy-acetylene welding. There were sometimes as many as 3000 pits in one length (23 feet) of pipe. The pipe was recoated carefully and heavily with a good quality of burlap and a prepared bituminous substance applied hot. The completed coating was from $\frac{1}{2}$ to $\frac{3}{4}$

inch in thickness. After placing in the trench and calking and testing, the joints were carefully coated.

In order to prevent, if possible, the destruction of the pipe in the future from the same cause, pieces of cast iron were placed outside the pipe at a distance of one to two feet and these were attached to the pipe by heavy copper cables. These protection devices have been in use for about four years and recent examinations indicate that so far they have been successful, i.e. the cast iron shows marked signs of deterioration whereas the steel pipe appears to be unharmed.

Corrosion and incrustation: Except for the effect of electrolysis noted above, neither the steel or cast iron pipe show any signs of corrosion, rust or deterioration. In fact on removal of the coating the steel pipe appears to be the same as when it came from the factory.

During the 3½ or more years that the various pipes have been in operation only the thinnest coating or incrustation has been collected; about as thick as a sheet of writing paper. This is very different from the Hadrian water which coated the pipes very rapidly.

Leakage and unaccounted for loss: Great care is taken to prevent losses from leakage. Two principal means are employed to detect leakage. The first means is a daily inspection of the minimum consumption as recorded by the meters. By this means a broken pipe which discharged into a sewer was detected within 24 hours. The second means consists of fortnightly inspections of all parts of the system that crosses sewers or other large conduits. Three leaks were discovered by this means in 1934.

An approximate criterion of the leakage of the system may be obtained by comparing the minimum hourly consumption with the various quantities which good practice considers as allowable leakage.

During 1934 the minimum hourly consumption was 200 m^3. This quantity includes the leakage and also the water used during this hour for bakeries, factories working nights, etc. For comparison, a leakage of 250 gallons per 24 hours per inch of diameter of pipe per mile of pipe is considered by one authority[3] as the upper limit of leakage for a well maintained system. 200 m^3 per hour in the Athens distribution system is equivalent to 456 gallons per day per inch of diameter per mile of pipe. The excess over the allowable quantity of 250 gallons can readily be accounted for as water used. Another

[3] Manual of American Water Works Practice, page 293.

authority[4] gives a maximum leakage of 1000 gallons per day per mile. 200 m^3 per hour = 2113 gallons per day per mile. A third authority[5] gives 3000 gallons per day per mile or an allowable leakage 50 percent in excess of the minimum hourly flow.

A still further check on leakage is the determination of the unaccounted for water. This can only be determined after a considerable lapse of time, say one year, unless unusual measures are taken.

During 1934, a total of 15,608,000 cubic meters of water was sent through the mains. This quantity is measured by Venturi meters. Of this amount only 1,566,000 does not pass through some consumers' meter or could not be accurately estimated. A certain part of this latter quantity, estimated at 316,000 cubic meters, is sold to about 1,170 flat rate customers. This leaves a total of 1,250,000 cubic meters or only 8 percent of the total which is not accounted for.

From all of the above we consider that the system is in very good condition and that leakage is at a minimum.

Installation of new pipe: During the past 3 years the system has been extended by the addition of 60 miles of pipe line or 12½ percent of the length of the system completed by the Construction Company. All new pipes are carefully backfilled with screened earth and the backfill is thoroughly settled or puddled with a liberal quantity of water. Before covering the joints, the pipe is tested with water under a pressure in excess of one and one half times the static pressure. The pipe is then flushed and heavily chlorinated with a portable chlorinator. Frequent samples of water are taken and tested and this continues until the pipe is free from pollution. Careful and permanent records are made of all of the tests noted above.

Valves: All ordinary valves in the system (6,650) are inspected and opened and closed at least once a month. During 1933, 844 leaking stuffing boxes were repaired and during 1934, 787. Forty valve stems were replaced in 1933 and 26 in 1934. Sixty-nine valve bodies were replaced in 1933 and 33 in 1934. Inasmuch as the number of repairs is decreasing it would seem as though the weaker parts are gradually being discovered and eliminated. The valves are not first class in quality but serve their purpose reasonably well.

There are 58 pressure reducing valves varying in size from 16 to 4 inches. A small gang inspects these valves regularly once a week to see that they are functioning properly and to make any necessary

[4] Journal, March, 1932, page 364.
[5] Journal, April, 1932, page 505.

repairs. These valves have functioned very satisfactorily. The only time that pressure is not reduced is when no water is passing through.

Float valves: There are 7 float valves for controlling the level in the small pressure reducing tanks. There was a conside. ble amount of difficulty experienced in adjusting these valves so that they would not cause water hammer in the pipes. Eventually, it was found necessary to put a gate valve in front of the float valve for approximate regulation of the flow by hand. They are inspected and regulated once a week.

House Connections: Pipes: Up to the end of 1934, 174 miles of pipe have been used for house connections. 91.6 percent is flexible copper tubing and the remainder lead pipe approximating the "double extra heavy" grade. At the present time, in Greece, a service constructed with copper tubing costs about one half as much as a service constructed with lead pipe. A record has been kept of the number of failures of services and this shows that the average number of failures per year for the past three years are 1 per 1000 for copper pipe and 59 per 1000 for lead pipe. From this it may be noted that flexible copper tubing is an excellent material for making satisfactory house connections. It is simple, inexpensive and easy to place and as, after 5 years of service, it shows no signs of deterioration it should last for a long time.

Water Meters: 71,260 meters have been installed. 96.9 percent are $\frac{5}{8}$, 2.2 percent are $\frac{3}{4}$, 0.8 percent are 1 and 0.1 percent are $1\frac{1}{2}$ to 6 inches. Nearly 74.6 percent are of the rotating disc type and are of American manufacture. The remainder are of the oscillating piston type and are of German and French manufacture. In general, both types are satisfactory.

A test of the two types of meters is being carried out.

14,000 m^3 have been run through eight $\frac{5}{8}$-inch meters of various makes. These tests, while not conclusive, appear to indicate that the rotating disc type maintains its accuracy better than the oscillating piston. The loss in weight of the piston after passing 14,000 m^3 through a $\frac{5}{8}$-inch meter is less for the disc type than for any of the oscillating piston type being 0.1482 grains for the rotating disc against 0.1645, 0.6678 and 0.9767 for three types of oscillating piston; or in percentage, the loss of weight is 0.31 percent against 0.72 percent, 1.67 percent and 4.25 percent. A careful record has been kept of the number of spare parts required to keep the various types

of meters in successful operating condition and it is found that fewer spares are required for the disc type, giving full consideration to the length of time the meters have been in operation.

In general, meters are tested by the meter testing department, whenever there are complaints and eventually it is planned that all meters will be overhauled at regular intervals, say 5 to 7 years, but this has not yet been put into effect. During the past year, 4,847 were tested, cleaned, repaired, etc.

During the first year there was a great popular outcry against the meters or so called "clocks," but this has practically ceased possibly due to the following causes: familiarity, education by means of pamphlets delivered with the quarterly bills and immediate test of meters declared to be faulty, usually by connecting a test meter in series with the alleged defective meter in the presence of the consumer. The gasoline or petrol tin found throughout the Near East and used as a measure has also convinced many customers of the accuracy of the meter.

COMMERCIAL

The various activities of the Company apart from those connected directly with the collection and distribution of water are grouped under this heading. These include the very important functions of billing and collecting, disbursement of funds, purchasing, storekeeping, transportation and employees welfare.

Billing and Collecting. As the payment for nearly all of the consumption is on a volumetric basis, the initial step in billing the customer is the reading of the meter. All meters are read monthly with the exception of very large consumers where meters are read weekly or oftener. In general the meter reader is kept in one district so that he gradually becomes acquainted with the customers. On an average the meter reader can read about 150 meters per day. All bills are prepared in the central office and are of the usual type of public service bill with two stubs. The bill indicates the previous and present meter reading, total cubic meters, various taxes and special charges including meter rent and total bill. All bills are made out on a European billing machine and addressed on an American machine. One stub is kept in the office while the other remains attached to the bill until it is paid. Bills are rendered quarterly for general domestic consumers and monthly for industrial and other large consumers. The cycle plan is in use for meter reading, billing

and collections and there is an even flow of work throughout the year. All payments are made at the main or branch offices of the Company or at a few specially designated banks and stores. There are no collectors. When bills are not paid promptly the service is disconnected and remains so until the bill has been paid together with a small charge for turning on water. Ninety-three percent of the customers pay regularly and only 7 percent have the water cut off. This percentage of cut-offs is steadily decreasing. Occupants of all separate apartments are compelled to pay for at least two cubic meters per month whether they are connected or not.

The price of water is 12 drachmas[6] per cubic meter for the first 15 cubic meters; 10 drachmas for 16 to 30; 8 drachmas for 31 to 100 and 5 drachmas for more than 100 cubic meters. There are special rates for industry, philanthropy, gardens and the government and municipal establishments. The average receipts from all water sold during 1934 was 7.70 drachmas per cubic meter and from all water delivered 6.45 drachmas per cubic meter.

General metered consumers take 48.2 percent of all the water delivered, at an average price of 11.10 drachmas per cubic meter. State and municipal consumers take 16.7 percent. Industrial 11.6 percent, various special paying consumers 6.8 percent. Free water including flushing and street sprinkling 8.7 percent leaving only 8 percent of all of the water delivered as unaccounted for.

Disbursements: 61 percent of the total operating expenses is for salaries and wages, the remainder being for material, rent, etc.

The total operating expenses are divided over the work as follows:

	percent
Collecting and storing water	4.73
Pumping	3.30
Purification	5.56
Transmission and distribution	25.99
Stores and transportation	7.42
Commercial	19.75
Rent, advertising and insurance	3.78
Legal Expense, Taxes, Bonus	6.96
Depreciation	8.70
Miscellaneous and Administration	13.81
Total	100.00

Purchasing and storekeeping. All materials, supplies, etc. are purchased by a separate department. In general the greatest care is

[6] One drachma equals approximately 1 cent.

taken to search for the most satisfactory combination of price, quality and delivery but unfortunately price is frequently the ruling consideration. All materials are carefully stored and delivered by careful weight and measure only against receipt. Tools are issued to the workman against his brass pay check. He must return the tools or he cannot get his pay. The result of all of this painstaking storekeeping is that quarterly physical inventories practically check the book inventories. Thus, in 1934, the total value of materials, tools etc. charged to stock was nearly 22 million drachmas. The physical inventory showed a book overage of a little more than 2.000 drachmas. The discrepancy is therefore 0.00964 percent. One reason for this satisfactory result is that the value of material is high compared to labor and therefore refinement in storekeeping is not an extravagance.

Useless material is collected and sold at auction from time to time. This not only reduces the required storage space but is a source of income. During the past year $8,000 were collected in this way. The material includes construction equipment which had been used in building the works and is no longer needed.

Transportation. Transportation consists of 1 large and 6 small passenger automobiles, 5, 1½ ton trucks and 2, ½ ton trucks. The direct cost for operating these cars not including depreciation is about 5½ cents per mile. The small passenger cars average about 3.8 cents. This includes chauffeur's wages, repairs and all supplies.

Insurance. Five types of insurance are carried. These include the usual automobile, fire and theft and fidelity as well as two rather unusual forms of insurance. The first is intended to cover losses occasioned by breaks in the system. And the second, losses resulting from defects in the system other than breaks. So far these last two types of insurance have proved very successful as the Insurance Company seems to be more successful in defending our case in court or in coming to an amicable settlement with the claimant than we would be.

Personnel. A little over 1,000 persons are engaged continuously. About 750 of them are laborers, plumbers, handy men, watchmen, etc. Medical assistance is given to all employees and their families. As yet there is no provision for old age pensions, but this is expected to come before long. The employees have an association which is recognized by law and have certain rights concerning employment and promotion.

ACKNOWLEDGMENT

The operation of the water supply is under the control of the Greek Government. In general this control consists in fixing water rates, approving large extensions and purchases, investigating certain types of complaints and auditing the Company's books. The spirit of good will and coöperation in which the various officials of the Government have carried out their difficult task deserves especial mention.

Credit for the systematic and painstaking collection of statistics of all of the various phases of the Company's activities is due to Mr. Georgalas, Mr. Nestorides and Mr. Farrell and to all of their collaborators who have so ably assisted them. The information given in the preceding notes is only the briefest possible resumé of the data on hand.

Finally I wish to express my wholehearted admiration for the manner in which all of the personnel of the Company have tackled the difficult task of operating an entirely new water system and for the success which has been theirs up to this time.

AMMONIA DOSAGE IN AMMONIA-CHLORINE TREATMENT OF WATER

By M. L. Koshkin

(Chief, Department of Experimental Hygiene of the Psychoneurological Institute, Kharkov, U. S. S. R.)

The method of ammonia-chlorine treatment of water holds a prominent place among the various modern methods of water sterilization. Ammoniation produces a certain influence on the various processes of chlorination and renders it preferable to ordinary chlorine-treatment in many respects. In previous study we experimentally ascertained the positive qualities of this method, described by several authors (McAmis, Gerstein, Baker and Schmelkes, Cox, Hinman and Beeson, Goehring, Lawrence, Ellms, Spaulding, Green, Braidech, Harold, Ruth, Enslow, A. Matheson, De Laporte, Dodd, Weir Smith, Ely, Amsbary, Caughey, Ehrhardt, Mustard, Capen, Keit, Hough, Irwin, Olszewski). In perfect agreement with other investigators we succeeded in preventing chlorophenol tastes and in increasing the bactericidal action of chlorine by means of ammonia compounds. We noticed that the effect of ammoniation changes according to the conditions in which chlorine-treatment is performed. Having systematically studied the influence of temperature and natural light on this process we proved that the chlorine absorption power increases with their rise and that the resulting changes are not only quantitative, but qualitative as well.

We studied also the importance of pH for water ammoniation and demonstrated the fact that the effect of ammonia compounds added to the water is best with pH approaching 7.0 or even with a higher pH value.

The ammonia-chlorine process is accompanied by a decrease in the chlorine absorption power in the presence of ammonia or ammonia-compounds. We could prove that the decrease in the chlorine absorption power of the water varies in accordance various admixtures. When vegetable matter (hay- or peat-moss decoction) is contained in the water the chlorine absorption power is greatly diminished by the addition of ammonia (becoming 2 or 3 times smaller); an even

greater decrease of the chlorine absorption power (attaining 1/80 to to 1/110 of the previous value) is observed in water containing phenol (0.05–0.2 mgm. per 100 ccm); the presence of animal matter in the water but slightly reduces its chlorine absorption power. (Special investigations are under way in our laboratory with a view to elucidating the nature of this phenomenon.)

The choice of the right dose of ammonia is of the utmost importance for the effect of preammoniation. The authors disagree on this point. At present ammonia is measured according to the weight of chlorine used. The optimal ratio between ammonia and chlorine found by various authors differed, varying from 1:10 to 1:2 (NH_3+ :Cl). Evidently such a difference in the optimal dose of ammonia was the result of the conditions under which ammoniation was performed by the different authors. In rating the ammonia dose to the chlorine no attention was paid to the admixtures and the other properties of water to be treated, whereas this factor is of no less importance for the choice of ammonia-dose than the quantity of chlorine used. Ammonia was generally dosed under laboratory conditions by investigators and in practical water treatment the workers of stations acted on empirical principles, and in every individual case such a ratio of ammonia-chlorine was chosen which yielded the best results under the conditions then existing. The empirical choice of ammonia dose is difficult and does not always warrant the best results. At the same time the determination of the ammonia dose is of crucial moment in the method of ammonia-chlorine treatment. If the dose be too small it may fail in preventing unpleasant tastes and odors, whereas too great an increase may weaken the bactericidal action of chlorine. Most interesting data on the interdependency between the ratio of ammonia, chlorine and phenol are cited by Braidech. By increasing the phenol dose from 0.05 to 1 mgm. per litre water Braidech obtained different results depending on the quantity of phenol used, while ammonia (0.06 mgm.) and chlorine (0.36 mgm.) content remained constant. In the presence of 0.05–0.1 mgm. phenol no chlorphenol taste was noticeable; 0.2 phenol produced a chlorophenol taste which increased with the dose of phenol when the author used a dose of ammonia three times as large as the former one; even 1.0 mgm. phenol did not produce any taste under the same conditions. At the same time Braidech proved that chlorophenol taste becomes stronger when the dose of chlorine rises, while phenol and ammonia are used in the same quantities.

In our studies of the ammoniation process the dosing of ammonia in

accordance to chlorine did not give us favorable results; the effective dosage of ammonia often proved higher and sometimes lower than the amount of chlorine determined beforehand. All these results demonstrate that the method of determination of the ammonia dose according to chlorine dose does not yield satisfactory results. This necessitated the finding of another method fit to replace the old one. We considered this all the more necessary as the properties of the water were not considered at all when the amount of ammonia was chosen in accordance with the chlorine dose, although the chemical and physico-chemical processes occurring in the water during ammonia-chlorine treatment do not only take place between ammonia and chlorine, but, evidently, between ammonia and other admixtures the water as well.

According to Schmelkes ammoniation decreases the oxidation potential of chlorine so that oxidation by chlorine diminishes and formation of chlorophenolic compounds is rendered more difficult. We think that ammonia not only acts on chlorine, but evidently also on the admixtures of the water, lowering their chemical activity. Only in this way is it possible to account for all the phenomena occurring in ammonised water when it is subjected to the action of chlorine. Therefore, it is necessary not only to be guided by the dose of chlorine, when determining the necessary amount of ammonia, but in the same measure also by the admixtures of the water and by its other properties.

CONTROLLING FACTORS IN AMMONIA DOSAGE

We pursued the following aim: (1) to find out in what degree the determination of ammonia dosage according to the quantity of chlorine used answers its purpose; (2) What conditions have a bearing on the dosage of ammonia; (3) to elaborate a method for determination of the ammonia dose according to the admixtures and other properties of the water and to the dose of chlorine.

In this work we used the following method: experiments were made with tap-water[1] with a pH 7.0–7.2. The results obtained with tap-water were not different from those obtained with distilled water, the only difference being the higher chlorine absorption power of tap-water. As organic matter added to the water meat-peptone-broth and hay decoction[2] were used.

[1] The chemical composition of the water is cited in our study published in Zeitsch. f. Hyg., Bd. 114, H 3; 4.

[2] The preparation of hay-decoction is described in the same study.

Ammoniation of the water was performed by means of an ammonia solution used 10 to 20 seconds before chlorine was added.

In the experiences with determination of bactericidal action of chlorine, used with ammoniation, besides the aforementioned organic matter a 24 hour-Coli commun culture in agar was added (5,000,000 organisms according to standard emulsion, to 1 litre tap water). The method was the same as in our preceding studies on ammonia-chlorine treatment. In order to determine what influence the ammonia-dose would have on the chlorine absorption power of the water, experiments were made in which the dose of ammonia and the amount of organic matter varied. With the view to rendering the conditions of chlorine absorption equal, this experiment was

TABLE 1

Chlorine absorption power with various doses of ammonia

DOSE OF AMMONIA	AMOUNT OF HAY-DECOCTION	
	0.5 cc.	2.5 cc.
None	0.624	1.263
0.006	0.546	1.089
0.012	0.423	0.956
0.024	0.319	0.887
0.048	0.319	0.751
0.096	0.251	0.578
0.144	0.251	0.526
0.384	0.226	0.526
0.768	0.227	0.508
1.536	0.227	0.508

performed in the same way as analogous experiments in our previous studies, i.e. chlorine was used in such an amount that after a 10 minute contact of chlorine and water the residual unsaturated chlorine amounted to 0.5–0.6 mgm. Under these conditions the chlorine absorption power of water with hay-decoction was determined in the experiments shown in table 1.

These experiments show that ammonia lowers the chlorine absorption power of water with hay decoction so that it became 2 or 2.5 times smaller than formerly, just as had been the case in our foregoing investigations. A certain dose of ammonia was wanted to obtain the lowest limit. The diminution of chlorine absorption power was considerable in the presence of a small amount of am-

monia; while the latter rose the chlorine absorption power continued to drop and attained its lowest limit in the presence of a certain dose of ammonia which we shall hence forward designate by the term "lowest limit dose." If the amount of ammonia was diminished still more, this did not have the same effect and if it did decrease the chlorine absorption of the water, it did so only in a minor degree. For 0.5 cc. hay-decoction this "lowest limit dose" of ammonia amounted to 0.096 mgm.; if the ammonia dose was increased still more the chlorine absorption power of the water dropped but slightly (if the dose of ammonia equalled 0.384 mgm. the chlorine absorption power attained 0.024 mgm. chlorine).

In experiments with 2.5 cc. of hay decoction the lowest limit dose of ammonia amounted to 0.144 mgm. By raising the amount of hay decoction the lowest limit dose of ammonia was thus increased. These experiments prove on the one hand that chlorine absorption power depends on the amount of ammonia present and shows on the other hand the dependence between the ammonia dose and the admixtures to the water. The most important practical results of these experiments consisted in the discovery of the lowest limit dose of ammonia which brought about the most important diminution of chlorine absorption power.

In order to determine what significance the ammonia dose has in preventing chlorophenol tastes and odors (the phenol-concentration being constant), experiments were made which are demonstrated in table 2. In these experiments the chlorine dose amounted to 0.144 mgm. and 0.2 cc. of meat-peptone-broth were added as well.

Tastes and odors were determined during a period of 48 hours following the beginning of the experiment. Table 2 shows that the dose of ammonia necessary for the prevention of chlorophenol tastes and odors in the presence of a constant dose of chlorine rises when the concentration of phenol is raised, so that in the presence of 1:40 million phenol the amount of ammonia necessary to eliminate odors and smells entirely, amounted to 0.42 mgm.; with a concentration of phenol 1:20 million 0.91 mgm. ammonia was necessary to produce the same effect; if there was a concentration of phenol = 1:10 million then 1.67 mgm. ammonia were necessary. When we raised the phenol concentration over the values indicated in table 2 we could ascertain that a further increase of phenol-concentration also demanded an increase of the ammonia dose in order to prevent chlorophenol taste and odor. In the experiment here described the dose of chlorine

remained constant while the dose of ammonia wanted for preventing of chlorophenol tastes and odor rose independently of the chlorine dose; according to the phenol-concentration present the ammonia dose could be either below the chlorine dose or 2 or even 4 times greater than the same. In this instance the determination of the ammonia dose according to the chlorine dose could prove inefficient.

TABLE 2

Interdependence between the concentration of phenol and the dose of ammonia

NUMBER	CONCENTRATION OF PHENOL IN MILLION PARTS OF WATER	NH_2 DOSE	RESIDUAL Cl	RESULTS					
				Immediate		After 24 hours		After 48 hours	
				Odor	Taste	Odor	Taste	Odor	Taste
		mgm.	*mgm.*						
1	1:40	none	0.063	int.	int.	int.	int.	int.	int.
1	1:40	0.01	0.126	dst.	str.	dst.	str.	dst.	str.
3	1:40	0.07	0.126	h.n.	dst.	h.n.	dst.	h.n.	dst.
4	1:40	0.14	0.142	none	sl.	none	sl.	none	sl.
5	1:40	0.28	0.126	none	none	none	sl.	none	sl.
6	1:40	0.42	0.142	none	none	none	none	none	none
7	1:20	0.07	0.079	str.	int.	str.	int.	str.	int.
8	1:20	0.14	0.079	sl.	int.	sl.	int.	sl.	int.
9	1:20	0.28	0.111	dst.	sl.	dst.	sl.	dst.	sl.
10	1:20	0.42	0.111	none	dst.	none	dst.	none	sl.
11	1:20	0.66	0.111	none	none	none	none	none	h.n.
12	1:20	0.91	0.126	none	none	none	none	none	none
13	1:10	0.28	0.079	str.	int.	str.	int.	str.	int.
14	1:10	0.42	0.063	dst.	str.	dst.	str.	dst.	str.
15	1:10	0.56	0.063	none	dst.	none	dst.	h.n.	dst.
16	1:10	0.84	0.079	none	dst.	none	dst.	none	dst.
17	1:10	1.32	0.126	none	dst.	none	dst.	none	dst.
18	1:10	1.67	0.126	none	none	none	none	none	none

Conventional signs: int. = intense odor or taste; str. = strong; dist. = distinct; sl. = slight; h.n. = hardly noticeable.

The dose of ammonia capable of preventing tastes and odors also varies depending on the chlorine dose used. This is demonstrated by the experiment in table 3. There the concentration of phenol amounted to 1:20 million, and 0.2 cc. of meat-peptone broth was added. The results of this experiment showed that the amount of ammonia preventing tastes and odors rises with the increase of the amount of chlorine; in the presence of 0.21 mgm. chlorine 0.6 mgm.

ammonia prevented taste and odor; when the chlorine dose was increased to 0.53 it was necessary to add 1.15 mgm. ammonia to attain the same end, and in the presence of 0.95 mgm. chlorine 1.49 mgm. ammonia had to be added to produce the same effect.

Table 3 also shows that the ammonia-dosage chosen according to the chlorine amount,—as is the general way of dosing ammonia, would not have the desired effect in this instance.

It is of the utmost importance to choose the right dose of ammonia in cases when the reaction of the water is slightly acid (pH 6.6–5.8). It is well known that chlorophenol tastes and odors are difficult to

TABLE 3

Interdependence between chlorine dose and ammonia dose

NUMBER	DOSE OF Cl	DOSE OF AMMONIA	RESIDUAL Cl	RESULTS					
				Immediate		After 24 hours		After 48 hours	
				Odor	Taste	Odor	Taste	Odor	Taste
	mgm.	*mgm.*	*mgm.*						
1	0.21	0.34	0.031	dst.	str.	dst.	str.	dst.	str.
2	0.21	0.46	0.031	dst.	sl.	dst.	sl.	dst.	sl.
3	0.21	0.57	0.046	none	none	none	none	none	h.n.
4	0.21	0.69	0.031	none	none	none	none	none	none
5	0.53	0.46	0.126	str.	int.	str.	int.	str.	int.
6	0.53	0.69	0.112	str.	int.	str.	int.	str.	int.
7	0.53	0.92	0.143	dst.	str.	dst.	str.	dst.	str.
8	0.53	1.15	0.143	none	none	none	none	none	none
9	0.95	0.92	0.301	sl.	str.	sl.	str.	sl.	str.
10	0.95	1.15	0.318	dst.	sl.	dst.	sl.	dst.	sl.
11	0.95	1.49	0.318	none	none	none	none	none	none

prevent by a means of ammoniation in water with an acid reaction. If the reaction of the water is strongly acid very great amounts of ammonia must be used to eliminate tastes and odors, so that amoniation is rendered practically difficult to realise; in the presence of a slightly acid reaction a certain raising of the ammonia dose may yield good results. Wishing to investigate this fact we performed an experiment with water having a pH = 6.7 and 7.2, the phenol concentration being 1:10 million, the chlorine dose 0.39 and that of organic substances (hay decoction) 0.2 cc.

It is clear from table 4 that the dose of chlorine being constant, the amount of ammonia necessary for prevention of taste and odor

varied in accordance with the reaction of the water. When the pH was 7.2 chlorophenol tastes and odors were obviated by an amount of ammonia lower than the amount of chlorine (0.3 mgm.)—while the chlorine dose was the same as before, whereas with water having a pH 6.7–0.81 mgm. ammonia was required for the same end, that is, an amount 2.5 times as great as that of the chlorine dose. Thus dosing ammonia according to the amount of chlorine present would have been ineffective.

All the experiments here described demonstrate that, when the dose of ammonia is fixed in accordance with the dose of chlorine, the amount of the former is sometimes so greatly diminished as to make ammoniation ineffective. When the aim of ammoniation is to pre-

TABLE 4

Ammonia dose with different pH of the water

NUMBER	pH OF WATER	DOSE OF AMMONIA	RESIDUAL Cl	RESULTS					
				Immediate		After 24 hours		After 48 hours	
				Odor	Taste	Odor	Taste	Odor	Taste
		mgm.	*mgm.*						
1	7.2	0.12	0.119	dst.	str.	sl.	str.	sl.	str.
2	7.2	0.23	0.171	none	dst.	h.n.	dst.	h.n.	dst.
3	7.2	0.35	0.171	none	none	none	none	none	none
4	6.7	0.12	0.102	int.	int.	int.	int.	int.	int.
5	6.7	0.23	0.119	str.	int.	str.	int.	str.	int.
6	6.7	0.58	0.154	dst.	str.	dst.	str.	dst.	str.
7	6.7	0.69	0.154	none	dst.	none	dst.	none	dst.
8	6.7	0.81	0.171	none	none	none	none	none	none

vent chlorphenol taste and odor and when it is desirable to cause active chlorine to remain for a longer spell of time in the water, a decrease of the ammonia dose has an untoward effect; on the contrary an increase of the amount of ammonia has favorable results. But if the dose of ammonia is too greatly raised the result may be a weakening of the bactericidal action of chlorine. This also forces us to be very careful in dosing ammonia.

Wishing to find out what significance the ammonia dose has for the bactericidal action of chlorine a series of experiments were performed, and the results are shown in table 5.

The method of the experiments was identical to the one just described. The doses of chlorine amounted to 0.465 mgm. and 0.691

mgm.; that of organic matter (hay decoction) equaled 0.2 cc.; the material was sown in Petri plates, 15 minutes, 30 minutes, 1 hour, 2 hours and 3 hours after contact with chlorine and the water under investigation.

In comparing the bactericidal action of chlorine alone and chlorine in connection with different doses of ammonia the results of table 5 were obtained. A certain lag in the action of chlorine when used with ammonia is demonstrable. When the dose of chlorine was 0.455 mgm. and that of ammonia 0.09 to 0.54 mgm. the lag in bactericidal action was only observed if the contact between chlorine and water

TABLE 5

Bactericidal action of chlorine in presence of various doses of ammonia

NUMBER	DOSE OF AMMONIA	DOSE OF Cl	RESIDUAL Cl	COUNT OF COLONIES AFTER				
				15 minutes	30 minutes	1 hour	2 hours	3 hours
	mgm.	*mgm.*	*mgm.*					
1	No NH_3	0.465	0.048	101	11	8	5	10
2	0.009	0.465	0.095	112	34	23	2	0
3	0.27	0.465	0.184	142	53	50	0	0
4	0.84	0.465	0.184	189	43	43	0	0
5	1.08	0.465	0.184	173	67	1	0	0
6	2.16	0.465	0.184	253	85	4	0	3
7	8.64	0.465	0.208	656	192	55	37	2
8	No NH_3	0.691	0.112	4	2	0	0	0
9	0.009	0.691	0.162	31	16	0	0	0
10	0.27	0.691	0.235	73	5	0	0	0
11	0.54	0.691	0.256	52	9	0	0	0
12	1.08	0.691	0.272	49	18	0	0	0
13	2.16	0.691	0.272	110	21	0	0	0
14	8.64	0.691	0.312	106	48	6	3	0

had been of short duration; if the time of contact was increased and reached 1 hour or even more, the bactericidal action of chlorine plus ammonia became even slightly superior to the action of chlorine alone; when the ammonia dose was 0.27 mgm. a lag of bactericidal action occurring after a short time of contact between water and chlorine was followed by a complete prevention of bacterial growth after 1 hour; the same dose of chlorine used without ammonia allowed colonies to appear even after 3 hours. The lag in bactericidal action increased with the rise of the ammonia dose. Not only did the number of colonies grown in the first periods of chlorination rise, but

the retardation of the bactericidal action of chlorine rose too. When the dose of ammonia amounted to 0.26 and 0.54 mg. and that of chlorine to 0.465 the retardation of bactericidal action was noticeable after 15–30 minutes contact between water and chlorine. This lag was observed even after an hour if the dose of ammonia amounted to 2.16 mgm. With a dose of 8.64 mgm. ammonia retardation was seen even after 2 hours.

When the amount of chlorine increased the bactericidal action rose in ammonised water and in water without ammoniation. In the presence of 0.691 mgm. chlorine retardation of the bactericidal action was apparent in ammonised water, but it was considerably less intense than the lag observed in the presence of 0.465 mgm. chlorine. It appeared again when the dose of ammonia was raised (8.64 mgm.). It must be mentioned, however, that a slight change in the ammonia dose had little influence on the bactericidal action of chlorine in the presence of 0.465 chlorine; 0.27 and 0.54 ammonia yielded well nigh the same results. When the dose of chlorine was a heavy one even considerable variations in the amount of ammonia (0.27 to 1.08 mgm.) had nearly the same results; ammonia in an amount of 8.64 mgm. and more, markedly diminished the bactericidal action of chlorine. Therefore, the dose of ammonia is important in attaining a bactericidal action. Doubtless the method of ammoniation may only be effective in all regards (preventing chlorophenol taste and odor, producing a sufficient bactericidal action, etc). when both ammonia and chlorine are used in the right doses. As to the dosage of chlorine we studied the optimal dosage of chlorine in ammoniation of water; the results obtained in our laboratory will be published in due time.

DETERMINATION OF NECESSARY DOSE

The method of determination of the necessary dose of ammonia which has been in use up to date, does not answer its purpose and ought to be altered. The method which we use in our laboratory for determining what dose of ammonia is necessary, is technically quite simple and its results are favorable.

We distinguish two possible instances in the determination of the ammonia dose; in ammoniation of water with medicinal tastes and when chlorinating and ammoniating water free from such taste and odor. Ammonia must be dosed in a most precise way when water with medicinal odor is being chlorinated. Even a slight lowering of

the ammonia dose may have untoward results. In this instance the chief object of ammoniation is to prevent offensive smells and tastes. Therefore, this condition must be borne in mind when fixing the dose of ammonia wanted to attain this end. This is accomplished when our method is used.

In this case we determine the dose of ammonia in the following way: a number of bottles is filled each with 1 litre of water under analysis; to this water first ammonia or salts of ammonia are added in increasing amounts; then after thorough mixing, the necessary dose of chlorine is added; the water in the bottles is well mixed once more and left standing for 1 hour. Than its taste and odor are evaluated. The smallest dose of ammonia capable of preventing tastes and odors must then be added; it is the dose of ammonia we wanted to find. Out of caution it is better to choose a medium dose between the smallest dose that prevented taste and odor and the next dose in size.

If for instance a series of bottles contained: 0.2, 0.4, 0.6, 0.8, 1.0 mgm. ammonia, and 0.6 mgm. were found to be the smallest dose that prevented the formation of tastes and odors, then we take 0.7 ammonia for ammoniation.

When raw water is treated by ammoniation and the testing of water might prove dangerous even after an hour's contact with chlorine—in the sense of a possible infection—one may take a little water from the experiment bottle, place it into a retort and boil and cool it before testing. Boiling does not eliminate the chlorophenol smell, but, on the contrary, it increases it. If the properties of the water allow it, it is best to test it and to determine its odor before boiling and after the same, as we do at present in our laboratory. Both tests are useful since testing of boiled water serves as a control of the exactness of the ammonia dosage and proves that the dose that was chosen is able to prevent chlorophenol taste and odor not only in raw water, but in boiled water as well. Likewise it is necessary to test raw water, for in some cases the water contains volatile substances producing unpleasant tastes and odors not to be eliminated by ammoniation, but disappearing during boiling. This might simulate a disappearance of tastes and odors due to ammoniation.

In cases when chlorine treatment does not produce medicinal tastes and odors and the chief object of ammoniation is to diminish the chlorine absorption power of the water, we determine the chlorine dose in another way. The aforementioned experiments have proved that slight changes in the amount of ammonia do not influence

the chlorine absorption power of the water. At the same time it was demonstrated that overdoses of ammonia may decrease the bactericidal action of chlorine and underdoses may result in an insufficient abatement of chlorine absorption power. In such instances care must be taken in dosing ammonia so as not to add too much or to diminish too considerably the amount of ammonia. We then determine the dose of ammonia by finding the value of the chlorine absorption power which we judge by means of ammonia. This is done by finding the "lowest limit dose" of ammonia. The chlorine absorption power is determined by means of the iodometric method in 1 litre water, as has been described in our former experiments. The smallest amount of ammonia which produced the greatest drop of chlorine absorption power is the dose of ammonia which we wanted to find.

TABLE 6

Dosing ammonia according to the chlorine absorption power

NUMBER	AMMONIA DOSE	CHLORINE ABSORPTION POWER
	mgm.	*mgm.*
1	0.1	0.47
2	0.2	0.31
3	0.4	0.31
4	0.8	0.31
5	1.0	0.23

Generally we use the following scheme for determining the dose of ammonia: 1 litre water is poured into each of 5 bottles and then 0.1, 0.2, 0.4, 0.8 and 1.0 mgm. ammonia are added; following this the chlorine is added and carefully mixed.

Table 6 represents an example of an experiment with chlorine absorption power, performed with a view to finding the lowest limit dose of ammonia.

In table 6 one may see that the minimal dose of ammonia which causes a marked depression of the chlorine absorption power was 0.2 mgm. This dose was considered as the necessary amount of ammonia. In increasing the dose of ammonia to 1.0 the chlorine absorption power was diminished again, but only in a slighter degree. In this instance it is best to take a dose of 0.2 mgm., for a slight decrease of chlorine absorption power does not justify an increase of the ammonia dose amounting to 5 times the former value, for the

bactericidal action of chlorine might suffer in consequence. As has been demonstrated in our foregoing experiments a slight diminution of the ammonia dose cannot have a bad effect on the results of chlorination. Therefore, it is best to take the greater dose when fixing the ammonia dose according to the scheme of table 6, if the case is uncertain, when the result of chlorine absorption power makes it difficult to decide which of the two ammonia doses ought to be chosen.

In our laboratory we have used this method for 2 years and are quite satisfied with the results. In practice it ought also to prove useful, but in order to judge of its practical value it is necessary first to test this method in water plants.

CONCLUSIONS

1. The chlorine absorption power of water (which in our cases contained hay-decoction), drops as the ammonia dose rises. A diminution of the chlorine absorption power proceeds up to a certain lowest limit dose of ammonia. When the amount of ammonia employed exceeds this limit no further decrease in the chlorine absorption power or only a slight one may be obtained.

2. The lowest limit dose of ammonia depends on the admixtures to the water and it changes when the latter are altered. In our experiments an increase in the amount of organic matter produced a rise of the lowest limit dose of ammonia.

3. The amount of ammonia which prevents chlorophenol taste and odor varies in accordance with the concentration of phenol and as the latter rises (the dose of chlorine remaining constant), a greater amount of ammonia is required.

4. The ammonia dose also depends on the chlorine dose. When the latter is increased a greater amount of ammonia must be used in order to prevent the appearance of chlorophenol taste and odor.

5. Water with a slightly acid reaction (pH 6.7) demands a greater amount of ammonia to prevent chlorophenol taste and odor than would have been necessary in the presence of the same dose of chlorine in water with a slightly alkaline reaction (pH. 7.2).

6. Ammonia retards the bactericidal effect of chlorine. A lag is observed after short periods of action (15–20 minutes). After longer periods (1–2 hours) the bactericidal action of chlorine-ammonia treatment exceeds the bactericidal power of chlorine used alone. But if too great amounts of ammonia were used (in our experiments

8.64 mgm.) this resulted sometimes in a considerable lag of the bactericidal action, still felt after long periods of contact. Raising the dosage of chlorine it is possible to increase its bactericidal action in the presence of large doses of ammonia thereby shortening the period of retardation of sterilization.

7. It was not possible to find a consistent proportion between the amount of ammonia able to prevent taste and odor and to preserve a sufficient bactericidal action and the corresponding dose of chlorine. According to the properties and admixtures of the water the dose of ammonia may be considerably below the dose of chlorine or 2 to 4 or even more times greater than the dose of chlorine.

8. The method of dosing ammonia according to the amount of chlorine, which is in common use at present, does not consider the admixtures and other properties of the water and cannot therefore guarantee the success of ammoniation in all instances.

The method of dosing ammonia which we have found is simple in its theoretical premises as well as in its technical use. The admixtures and other properties of the water are taken into consideration in this method which gave favorable results in laboratory experiments. It deserves to be studied in water plants so that its practical value might be determined.

REFERENCES

(1) McAmis, Jour. Amer. W. W. Assoc., 17, 1927.
(2) Harold, Jour. Royal Army Med. Corps, 45: 3, 4, 5, 6, 1926.
(3) Olszewski, Pharmezeutisch. Zentralhalle, 20, 1926; 27, 1927.
(4) Olszewski, Chemiker Zeitung, 28, 1927; 14, 1928.
(5) Hinman and Beeson, Jour. Amer. W. W. Assoc., 21: 12, 1929.
(6) Spaulding, Jour. Amer. W. W. Assoc., 21: 8, 1929.
(7) Cox, Jour. Amer. W. W. Assoc., 21: 12, 1929.
(8) Gerstein, Jour. Amer. W. W. Assoc., 23: 9, 1931.
(9) Baker and Schmelkes, Ammonia chlorine process and its application in the treatment of water. Monograph, 1931.
(10) Goehring, Jour. Amer. W. W. Assoc., 23: 9, 1931.
(11) Lawrence, Jour. Amer. W. W. Assoc., 23: 9, 1931.
(12) Ellms, Jour. Amer. W. W. Assoc., 23: 3, 1931.
(13) Braidech, Jour. Water Work Engineer, 433, 1931.
(14) Ruth, Jour. Amer. W. W. Assoc., 23: 3, 1931.
(15) Matheson, Jour. Amer. W. W. Assoc., 25: 5, 1933.
(16) De Laporte, Jour. Amer. W. W. Assoc., 25: 5, 1933.
(17) Caughey, Jour. Amer. W. W. Assoc., 25: 5, 1933.
(18) Mustard, Jour. Amer. W. W. Assoc., 25: 5, 1933.
(19) Keith, Jour. Amer. W. W. Assoc., 25: 8, 1933.

(20) SMITH, Water Works and Sewage, 80: 5, 1933.
(21) ELY, Jour. Amer. W. W. Assoc., 25: 9, 1933.
(22) AMSBARY, Ibid., 25: 9, 1933.
(23) EHRHART, Ibid., 25: 9, 1933.
(24) HOUGH, Ibid., 25: 2, 1933.
(25) SCHMELKES, Jour. Amer. W. W. Assoc., 25: 5, 1933.
(26) CAPEN, Water Works and Sewerage, 81: 1, 1934.
(27) KOSCHKIN, Zeitschr. f. Hygiene, Bd. 114 H3, Bd. 115 H1, Bd. 116 H1.
(28) IRWIN, Jour. Amer. W. W. Assoc., 26: 9, 1934.
(29) DODD, Jour. Amer. W. W. Assoc., 26: 6, 1934.
(30) KOSCHKIN UND SPEKTOR, Zeitsch. f. Hygiene, Bd. 116 H6, 1935.

ACCOMPLISHMENTS IN THE STATE SUPERVISION OF DAMS

By George W. Hawley

(*Deputy State Engineer in Charge of Dams, Sacramento, Calif.*)

The accomplishments of the State supervision of dams are so far reaching in their effect and of such inestimable benefit, direct and indirect, that their exact and complete appraisal can be properly made only with time. While it might be untimely to judge finally the success of the law governing the supervision of dams, which seeks to provide safety through State control, excellent progress has been made under it, and if the law and administering office are properly supported by the public and the legislature there will, without question, be a greater assurance of safety of dams in this State in the future than there has been in the past.

It is, however, believed appropriate and timely to present a brief résumé of our activities at this convention because it was in this community that our first and one of our most important actions was taken. Just five years ago this week the Los Angeles County Flood Control District filed application for approval of plans and specifications for a concrete gravity dam of unprecedented height then in construction at the Forks Site in San Gabriel Canyon. This application was, as you know, disapproved by the State Engineer on the grounds that the dam would, if built as proposed, be a menace to the populous San Gabriel Valley below. From that day to this a large part of our more important work has been south of the Tehachipis with a very considerable part of our time and effort devoted to the many continuing and complex problems of the San Gabriel Canyon.

It has well been stated that the design and construction of dams is an intriguing and fascinating adventure. This is true throughout California, and particularly so in southern California, because of the increasing necessity for the conservation and utilization of the State's most precious natural resource, water; because of the urgency of affording protection to life and property against the ravages of recurring devastating floods, and because of the large number of dams

of increasing importance and magnitude constructed and to be constructed—all imperative to the sustained economic and social development of our State.

In striving to attain optimum use and control of our water resources through the construction, operation and maintenance of dams and appurtenant structures, the potential menace of these works, if incompetently designed, improperly constructed or unsafely operated and maintained, must be recognized and guarded against. Dams, if not safe, are a great public hazard and therefore assurance that dams can and will be safely built is of vital importance to the State.

For the purposes of safeguarding life and property, the Legislature of 1929 invested the duty of and authority for the supervision of dams, with the exception of those owned by the United States, in the Department of Public Works to be exercised and administered by the State Engineer. This legislation, the most far reaching and competent of its kind to be enacted by any legislative body, has been held in a series of court decisions to be "constitutional in all its essential provisions as not only a proper, but as a necessary exercise of the police powers of the State." The police power of the State has been defined as "an attribute of sovereignty and on it depends the security of society, order, the life and health of the citizen, the comfort of existence, the enjoyment of private life and beneficial use of property." It is in conformity with this sovereign power that the "dam" law was formulated.

State control in the act as passed is limited to that necessary in the interests of safety and embraces three main features:

1. Inspection of existing dams;
2. Approval of plans and specifications for and inspection of construction of new dams, the enlargement, alteration, repair and removal of dams;
3. Supervision over maintenance and operation of all dams in so far as necessary to safeguard life and property.

Immediately after the law became effective and funds available, the State Engineer organized the department of dams as a branch of the Division of Water Resources of the Department of Public Works. Appreciative of the volume, importance, responsibility, and highly specialized nature of the work involved in administering the law, the personnel of the department was carefully selected on the basis of technical ability, experience, and other requisite qualifications for passing on the safety of dams. A complete procedure for reasonable

but firm enforcement of the law was worked out. Plans for a dam are first exhaustively analyzed by engineers of the office, then the structure itself, or the site in the case of a new dam, is carefully examined by engineers and geologists and such explorations, core drillings, etc. as are needed, are required and on the basis of complete information decision is reached regarding the safety of the plans, site or dam in accordance with best modern engineering practice. In case a dam presents unusual difficulties a consulting board of eminent geologists and engineers is appointed to report to the State Engineer on the safety features.

The personnel of the department, together with consultants, when necessary, has aggressively and intensively initiated and developed such studies, investigations, and examinations as have been necessary to pass competently upon the "safety features" of each dam and its appurtenant structures. I say "safety features" advisedly because, in many instances, demand has been made on the department, either directly or indirectly, to consider and advise on such matters as contractual relationships or controversies, economic feasibility, water rights, engineering design or construction not directly related to safety, selection by the applicant of personnel or consultants, adoption of one type of dam in preference to another, assumption of engineering responsibility, etc. In all such matters the department is wholly without authority, legal or professional, and the department stands unwilling to trangress on the activities and responsibilities of the engineering profession, to usurp or infringe on the rights and privileges of the interested parties, or to be in any way a party to any endeavor unrelated to safety.

It is the aim and endeavor of the department, scrupulously adhered to, to require that the personnel refrain from forming conclusions on the basis of local or personal influence, refrain from imposing unwarranted or dictatorial conditions beyond the requirements of safety, refrain from exerting unnecessary influence over construction, refrain from assuming engineering direction, and refrain from expressing opinion or influencing decision in connection with economic or engineering considerations such as choice of type, selection of site, etc. On the other hand the personnel stands willing at all times to discuss informally with the engineer, when acting for the applicant, controversial problems of design or construction which relate to the safety of the dam under consideration.

As provided for in the act, working agreements have been effected

with other jurisdictional agencies, such as the Federal Power Commission, the U. S. Department of Agriculture (Forest Service), the State Fish and Game Commission and the California Debris Commission, whereby effective and efficient supervision of dams under joint jurisdiction, may be exercised with a minimum of inconvenience to the owner.

It has been found advisable to maintain engineers resident in southern California in order to expedite our examination of existing dams, to supervise effectively work in progress, and to render a maximum of service to owners of dams in this section of the State.

The personnel and procedure of the department must, of necessity, permit extreme flexibility in order competently and readily to meet the wide scope of activities required as a basis for sound and proper determination of safety. This variation in duties results from the fact that:

1. The art of design and construction of dams which is in an evolutionary stage is highly technical and advancing with rapid strides.
2. There is a wide range in type and location of structures.
3. The variation in geological formations throughout the State makes the study of foundation conditions of each dam an independent but important problem.
4. The volume of work involved in passing on in excess of 700 dams necessitates efficient and energetic prosecution of the various activities.

The activities of the department have been directed along six general lines of endeavor:

1. Field examination and investigation of existing dams.
2. Geological examination and report on dam sites, both of existing and proposed dams.
3. Analytical study of the structural features of all dams and appurtenant works.
4. Hydrographic study of the drainage basin area tributary to each dam.
5. Hydraulic study of the spillway of each dam.
6. Competent and regular inspections of all new construction and repair work.
7. Continuing supervision of the maintenance and operation of existing dams.

The efforts of the department have been aggressively directed to an

examination and critical review of some 700 existing dams; to the issuance of orders directing that the necessary work be done to render those dams safe where unsatisfactory physical conditions obtain; to the review and analysis of plans and specifications for the construction or enlargement of 130 dams; to the supervision of repair of some 400 dams, and to supervision of maintenance and operation of all dams under jurisdiction. It might be of interest to note that the new construction before the office for action or acted upon involves an expenditure approaching $65,000,000 and will when completed increase the storage capacity of impounding reservoirs in the State by about 1,000,000 acre feet. The combined storage capacity of reservoirs under jurisdiction totals approximately 7 million acre feet and represents an investment of upwards of $160,000,000.

The dams under jurisdiction are of widely diversified types; namely, earthfill, rockfill, hydraulic fill, gravity, single arch, multiple arch, slab and buttress type, as well as many composite structures, and may be classified as follows:

Percent	*Type of dam*
63.6	Earth
10.7	Arch
9.5	Gravity
7.3	Rockfill
3.1	Timber
2.6	Multiple arch
2.3	Slab and buttress
0.9	Miscellaneous

During the period the law has been in force 829 applications for approval of existing dams have been filed with the department, 131 applications for construction of new dams or the enlargement of old dams, and 403 applications for the approval of repairs or alterations to existing dams. These latter have for the most part been filed in conformity with the findings of the department that repairs were necessary to put the structures in a safe operating condition.

Of the 829 dams originally filed upon by the owners approximately 700 were found after investigation to be in jurisdiction. When it is remembered that approximately 400 of these required repairs in order to be safely used, it is apparent that an actual need existed for the passage of legislation such as the dam act.

Furthermore, it must be recognized that a dam found to be safe under one set of conditions or at a particular time, will not con-

tinue to function as a safe structure under changed conditions unless it is properly maintained or altered to provide for safe use under such changed conditions or in the light of increased information and knowledge as to the characteristics of the dam. Consequently the necessity of a continued and intimate knowledge of each structure is of major importance for the assured safeguarding of life and property.

Of the 403 applications for repairs and alterations submitted, many of them covering more than one item, 245 included work on spillways; 92 work on repairing and replacing of outlet conduits; 52 work on strengthening and amplifying earth and rockfill dam sections; 31 work of decreasing leakage and seepage; 26 work on the faces of dams, either to prevent wave wash or replacements of timber facing on rockfills; 18 covered alteration work which would in effect remove the dam; 18 required work for strengthening the members of rigid structures; 14 required work on the foundations and the other 12 required miscellaneous repairs.

In effect, therefore, of all defective dams 60 percent were defective in the provisions for wasteways, either hydraulically or structurally. Consequently, it is apparent that in the past either this all important feature of safe dam construction has been given too little attention or maintenance of spillway structures has been sadly neglected.

With approximately 70 percent of the dams in the State earth and rockfill, requiring spillways for the safe passage of flood flows, and with many of the rigid type structures not designed for overpour, not only must the department give greater thought and study to the investigation of run-off and flood characteristics, but the engineering profession and dam owners should coöperate to the extent of assisting in establishing recording stations and keeping accurate flood flow records in order to make our knowledge of these very essential matters more complete. In the past most engineers and dam owners have been interested primarily in the total monthly or annual yield of a watershed, instead of in short time floods of maximum intensity which of necessity, in many instances, must govern in the design of adequate spillways in order to afford ample safeguard of the storage works, the water supply and the interests of those in the area lying below the dam.

An analysis of records discloses that a major number of dam failures has resulted from inadequate spillway provisions and from foundation or abutment insufficiencies. Where ordinary care has been exercised in design and construction, instances of failure within

the structure itself have been extremely rare. This indicates not that less thought and effort should be directed to technical design, but that more thought should be applied to geologic and hydraulic features.

Engineering technique, in the design and construction of dams and appurtenant structures, is in a progressive stage in which rapid advance is being made. Accelerated intensive analytical study, concerted research, and increasing knowledge of the materials and methods of construction are advancing the art of design and construction of dams, both structurally and hydraulically.

The highly technical features of dams, foundations and appurtenant structures such as spillways can, from the technical point of view, be most competently and profitably discussed after completion of our examination into the safety of the 770 odd dams and the completion of the several technical investigations which the State and other agencies have initiated or will shortly undertake. These investigations include such studies as the arch action in dams, uplift, internal stress distribution, the hydraulics of weirs, siphons, lateral flow spillways, etc. At this point it might be well to recite briefly some of the many technical problems with which we are confronted, many of which are in a controversial state and in connection with which many important data have been obtained and toward the solution of which energetic and comprehensive research is being directed.

Factors entering into the determination of the distribution and magnitude of stresses within concrete dams may be briefly mentioned with regard to each of the following types:

Arch Dams:

1. The application of the theorem of elasticity to the concrete arch.
2. The effect of tangential shearing forces.
3. The effect of torsional forces.
4. Temperature changes and shrinkage during curing.
5. Nonuniform variations of temperature within the structure.
6. Distribution of loading between arches and cantilever.
7. Bending and shear deflections in cantilever.
8. Distribution as determined after complete analysis by the method of trial loads.
9. Deformation of the foundation.

10. The effect of water soaking of concrete.
11. Alteration of stress distribution through the influence of Poisson's ratio.
12. Modulus of elasticity of concrete and its variation with stress, age and moisture content.
13. The design of gravity abutments of various types.
14. The quality of the foundation and its treatment.
15. Construction methods.

Multiple Arch Dams:

1. In arch design, considerations as have just been mentioned.
2. In buttress design—distribution of stresses, stability against sliding, overturning and buckling.
3. Determination of stresses due to seismological disturbance in the arches.

Gravity Dams:

1. The internal stresses within the dam, their distribution and magnitude and the stability of the section due to water load.
2. The problem of uplift at and below the base and along construction joints, and within the pores of the concrete and its effect on stress distribution and stability of the section.
3. Hydrodynamic and inertia effect of earthquakes.
4. Properties of mass concrete, under the effects of temperature, shrinkage and watersoaking.

Many of the considerations mentioned above under each class apply equally in the analysis or design of gravity, arch and multiple arch dams.

The following general comments are offered with regard to each of the following:

Earthfill Dams:

In the earthfill dams consideration must be given to the following points:

1. Slopes and crest dimensions.
2. Selection of proper freeboard.
3. Physical properties of materials.
4. Planes or zones of saturation.
5. Seepage and its effect on the stability of the fill.
6. Erosion due to wave action and precipitation, and its prevention.

7. Preparation of foundations.
8. Type and location of cut-off.
9. Methods of construction.
10. Types of facing.

Rockfill Dams:

In the rockfill dams the following points are given careful consideration:

1. Dimensional data of section.
2. Stability of section.
3. Impervious facing and cut-off.
4. Preparation of the foundation.
5. Character of rock composing the fill.
6. Leakage.
7. Proper freeboard.

Spillways:

Relative to spillways we have to consider:

1. The type,—whether weir, channel, overpour, siphon, lateral flow, vertical shaft or tunnel.
2. Coefficients of discharge to be used in the analysis.
3. The effect of entrained air on the discharge.
4. Nonuniform flow.
5. The hydraulic jump.
6. Erosion within the channel and below.

Hydrographic studies involve the following points for consideration:

1. Precipitation.
2. Run-off intensity and expectancy.
3. Period of concentration.
4. The watershed area, its elevation, surface geology, ground cover and shape.
5. Comparison of results with adjacent watershed of relative properties.
6. Effect of combined rainfall and melting snow.
7. Preparation and study of reservoir flow sheets.

It is essential that detailed geological and seismological studies be made.

While a large amount of extremely valuable technical study and research has been done, there still remains a pressing demand for searching investigation in many subjects and fields directly allied with the construction of dams, for example:—

Review, compilation and analysis of existing literature, data and

research related to the design and construction of dams, including all types of hydraulic structures.

Behavior of foundations and abutments under load or when subjected to seepage and saturation.

Functioning of structure under load.

Properties of construction materials.

Settlement and movements of dams.

Types of impervious facing.

Protection against erosion and scour.

Compilation and analysis of hydrographic data.

Hydraulic properties and efficiencies of spillways, outlets and weirs.

Studies of highly specialized subjects such as trial load method for stress determinations, analysis of framed structures, loads and stresses in conduits under fills and tunnel linings; the raising of gravity dams, best type of controlled outlets, junction between rolled and hydraulic fills.

Grouting of foundations, cut-offs, and contraction joints.

The department, in so far as may be possible and advisable, is endeavoring to program its methods and policies to the end that uniformity of procedure in accordance with accepted engineering practice be obtained. It seems desirable that early concerted action be taken by engineers in order to attain engineering accord on minimum criteria for the building of safe dams.

As already pointed out, it is believed that excellent progress has been made in supervising dams under the State law.

The attitude manifested by the public, owners of dams and engineers alike is convincing and reassuring as to the desirability and soundness of the law and its administration. The spirit of willingness to accept supervision, the desire to coöperate heartily, the valuable assistance rendered and the ready accession in complying with suggestions and recommendations of the department are indicative of the accord with which the legislation is being received.

In the light of our experiences and observations and based upon the comments of interested parties, immeasurable benefits have, because of State control, accrued to the public, the owners of dams and to the engineering profession.

From the public's points of view, State control allays inherent fear and apprehension of the safety of dams thereby eliminating or minimizing unjustified opposition to the building or maintaining of dams, obviates the expense of litigating against alleged unsafe dams,

centralizes and makes for uniform control of dams where one or more agencies have joint jurisdiction, and assures the public of unbiased and unprejudiced engineering opinion.

To the owners of dams, State supervision has been of value in that it mitigates against questionable claims of unsafety, brings to the owner's attention the necessity of doing certain work to render their dam safe of which they otherwise would have no knowledge, materially reduces the possible loss of their structure, including the impounded water, as well as the financial losses or damages entailed. State supervision often times results in a reduced operating expense to the owner, and in many instances results in securing for the owner competent engineering advice and the services of an experienced contractor that he would not otherwise obtain thereby resulting in a better job at a lesser cost.

And last but not least there inheres to the engineer many worthwhile advantages. There is and has been a marked trend on the part of the owner to employ experienced engineers to advise him in regard to his dam and to supervise his work, thus adding to the volume of engineering work and placing this important work in the hands of qualified engineers rather than unqualified parties for, in the past, dam building like other engineering work has been looked upon by some as a job any lay person could undertake. In many instances the engineering profession has benefited because of higher standards of design and construction and a more secure structure has thereby resulted. There has in part been brought together, for discussion and determination, conflicting points of view to the advantage of all concerned and a sound decision reached which without engineering direction might be decided by some financier, farmer, business man, partisan newspaper or more aggravated still, some elective official, each of whom might be wholly unqualified. There has been an increased appreciation, and confidence in the work of the engineer by the public. There has been an added incentive and willingness on the part of financial people and agencies to finance new construction as well as the rehabilitation of existing unsatisfactory structures. Because of increasing standards and the rapid strides made in the art of design and construction of dams there has been an acceleration in investigation and research and an incentive to obtain observed data on the behavior of existing structures. Under State supervision a centralized agency is established, where an ever increasing volume of data can be assembled pertaining to hydrography,

hydraulics, geology and seismology which can be correlated and made available in usable form. Pertinent physical and dimensional data and technical information are also made of record in connection with all types of structures much of which, if not on file, would be lost or unavailable.

In dam construction more than in any other construction perhaps, unforeseen and difficult situations arise, necessitating changes in plans, increasing the cost to the owners or even resulting in the abandonment of the dam. In such instances fullest coöperation is required among all interests involved. The State's responsibility in these situations is great, but more important is the fact that the State office must undertake the grave duty of assuring those living below a dam and the public generally that all reasonable safeguards have been taken in its construction and that lives and property will not be jeopardized because of the increasing tendency to build higher dams, of greater storage capacity, with greater attendant potential menace.

The mandatory duties placed upon the State Engineer in exercising jurisdiction over the dams in this State and the responsibility exacted of the State Engineer to the end that dams be constructed and maintained as safe structures, are tremendous in that they affect not only expenditure of millions of dollars by builders of dams, but involve in large degree safety of human life.

In the state supervision of dams, however, it should always be remembered that the advancement of any community in this State depends in large part upon the orderly development of its water resources through the construction of dams. This program to be most effective must not be retarded through over cautious and unwarranted functioning of the office having jurisdiction.

State supervision of dams, if properly and competently administered and if directed aggressively and solely to the proper requirements of safety, merits the support of the engineering profession, the public and the owners of dams.

(Presented before the California Section meeting, October 25, 1934.)

HANDLING OF DELINQUENT ACCOUNTS

By C. E. Thatcher

(Manager, Commercial Division, East Bay Municipal Utility District, Oakland, Calif.)

In dealing with this subject we shall first describe what the District provides in the way of conveniences for its consumers to enable them to pay their bills easily within the limit of the credit period and what is required of a consumer applying for water service, i.e., the establishment and maintenance of credit.

In addition to the four business offices the District maintains, thirty-seven agencies have been established in residence and business centers and urban and rural territory where bills may be paid. Thirty of the agencies are branch banks.

An applicant may establish and maintain his credit by a cash deposit, or otherwise, before the service is rendered. "Or otherwise" means that, if the applicant has (recently) been a consumer for one year or more at some other location and was not in arrears during the period service was rendered, credit is considered as established. These requirements apply to all applicants regardless of whether owner of the property or a tenant.

During the fiscal year ended June 30, 1934, 1,427,047 bills were paid as follows:

per cent	
55	at the counter of the business offices
19	by mail
9	to collectors and service men
16	at branch banks
1	at other agencies
100	

Consumers' bills are rendered monthly. On the face of the bill the following is printed:

This bill is due and payable upon its receipt by the consumer, and if not paid within 15 days thereafter, service is subject to discontinuance without further notice.

All bills are mailed to consumers. When a bill containing charges for one month is not paid within twenty days after it is mailed, (5 days' grace is allowed to cover delays in mail, etc.), it is classed as delinquent and an effort is made to collect it. The memorandum section of the bill is then taken out by a collector. If the collector is unable to get in touch with any one at the premises, a notice is left showing thereon the amount of the bill; notifying the consumer that it is past due and requesting prompt payment. The notice does not contain any threat of interruption of the service. When the collector *is* able to present the bill to the consumer, attention is called to its being past due and a polite request made for payment. This effort to collect might be considered a courtesy call and is an added convenience brought to the door of the consumer.

When a bill containing charges for two months is not paid within twenty days after it is mailed, it is then classed as delinquent and to the extent that the service may be subject to interruption in case payment is not made within a specified time. The memorandum section of the bill is then mailed with a notice pasted thereon informing the consumer that the bill must be paid on or before a specified date to avoid subjecting the service to interruption. The "specified date" inserted in the notice allows sufficient time for the memorandum bill and notice to reach the consumer so as to permit of a full five days' extension of credit, after its receipt, in which to make payment.

If, at the expiration of the five days' extension of credit, the bill remains unpaid, a special bill is prepared and presented at the premises for payment. If payment is not secured the service may be turned off and in a great many cases is.

Inability of the field representative to secure payment is due to a number of causes. The action to be taken is left entirely to him. He is on the ground and must be governed by existing conditions. It may be found that the person who actually pays the bill is not at home and will not be until the close of the day, or away for a longer period. If, in his opinion, the atmosphere about the premises indicates a well established home and he learns through conversation that both the water bill and notices have been received, also that those with whom he converses do not understand why the bill has not been paid, he may turn off the service.

Should there be sickness in the home or other conditions exist over which the consumer has no control, or unable to make payment on account of funds not available until a few days later, the service is

not interrupted. However, an understanding is had with the consumer or others at the time that if the bill is not paid within four or five days it will be necessary to turn off the service. A notice is left to that effect.

When it is found that there is no one at home, the field representative is governed pretty much by what has transpired in the past in attempts to secure payment of delinquent bills. If the records show that it has been necessary to make repeated calls at the premises with delinquent bills for two or three months in succession and failure to secure payment was due to being unable to see any one, the service is turned off. A notice is left, showing thereon the amount of the bill and stating the reason for the turn-off. If it is decided to not turn off the service a notice is left informing the consumer that payment must be given immediate attention to avoid interruption to the service. Generally, three to five days are allowed in which to make payment. If the consumer fails to remit within that time the usual treatment follows. Payment is secured or the service is turned off.

When a consumer assumes an indifferent attitude, and states that his bill will be paid when he sees fit to do so and at his convenience, the immediate turning off of the service results.

Some consumers will not answer the signal at the door and attempt to dodge and ignore the "water collector." An experienced collector is quick to sense such movements and the service is turned off. Observing this, the consumer comes out in the open and satisfactory results are generally obtained. It has often been said that: "There are tricks in all trades." The collector finds it necessary to spring a surprise trick now and then. Often strategy as well as diplomacy must be resorted to.

Extension of credit, granted beyond the usual twenty days, on bills containing two months' charges, creates a large number of bills containing three months' charges through the failure of consumers to pay before the expiration of the extended date and before final action can be taken by the District.

Bills containing charges for three months, when rendered are not mailed to consumers. The limit of credit has been reached. Such bills are taken out immediately for collection. Payment in full or a payment for the amount of the two oldest months must be secured or the service is turned off.

A consumer moving about within the District's boundary may request that the closing bill be mailed to the new location. This is

permissible. If the closing bill is not paid within twenty days from the date it is mailed it is classed as delinquent and an effort is made to secure payment. If the consumer fails to pay, further treatment of the bill is the same as the delinquent two month bills receive and which may result in an interruption to the service at the new location.

For the past three years it has been necessary to set aside rigid rules governing credit and collections, due to economic conditions. While vigilant and painstaking supervision has been constantly exercised, rules governing credit have been stretched to the limit and payments have been secured little by little. Satisfactory and well defined practices which were in force five or six years ago have practically been forgotten. Thousands of consumers classed as "good pay" then have dropped to a lower credit level and nearly every month drift into the delinquent group. In addition to this group other thousands have been forced into a "slow and doubtful pay" group. The treatment accorded under such conditions has at all times been applied with understanding and judgment. Almost every case has had to be weighed on its own merit and with due regard to the conditions affecting the consumer.

Consumers in business have suffered temporary set-backs; others who had been steadily employed for many years were forced into the unemployed ranks. The granting of an extension of credit and the arranging for partial payments of delinquent accounts appeared to be the only reasonable and fair way to deal with these unfortunate consumers. Due to these conditions the number of delinquent accounts handled monthly has increased considerably. This liberal treatment accorded consumers resulted in an increase in the field service and collection work. A greater number of calls have to be made and every effort must be put forth to assist the consumer.

The District has no charity fund and must deal with the authorized charity agencies. Therefore, to administer collection work effectively involving such delinquent accounts, it has been necessary to establish close, as well as reciprocal, relations with all municipal, county, state and federal agencies administering aid to indigents, other unfortunates and the unemployed, with the result that the District and the agencies have been greatly benefited by each other's findings and reports rendered. Many delinquent accounts bave been protected and partial payments received from time to time due to the information obtained through these agencies regarding the true status of the consumer receiving aid and the obtaining of employment.

Keeping everlastingly at it and exercising patience, even when there

has been only the slightest possibility of receiving a partial payment on a delinquent account has resulted in preventing an interruption to the service. It can be readily seen that the rules referred to in the foregoing have been stretched almost to the breaking point.

A few years ago established procedures, governing the handling of delinquent accounts, could have been referred to as fixed and fast rules having been made effective, intelligently applied and often automatic or mechanical. Conditions were different then. Ninety-five percent of the consumers could pay their bills promptly and did. But few became delinquent.

To describe what was accomplished in 1929 in establishing and enforcing air-tight collection rules, viz., requiring the payment of the monthly bill before the issuance of the bill for the succeeding month; turning off the service if not paid and producing the results desired by reducing the number of outstanding Consumers' delinquent accounts receivable to less than 0.5 percent of the number billed monthly, would, it is thought, be of but little interest because such rules could not now be made effective. Conditions changed. The District was quick to recognize the change and did not delay in modifying the rules governing collections and extended the credit period in order to lighten the consumers' burden and share with him. Good will and a kind and friendly feeling was created by such action. This modification of the rules resulted in the number of outstanding delinquent consumers' accounts receivable gradually increasing to 3½ percent of the total number billed monthly.

The District's percentage of loss due to cancelled accounts classed as doubtful and hopelessly uncollectible for the past two fiscal years, was 0.36 percent of the gross revenue billed regular water consumers. For the four prior fiscal years the percentage was a little less than 0.33. Even with the unusual financial condition of consumers to content with and the liberal extension of credit granted, there has been only a slight increase in cancelled accounts.

This condition is not considered bad or at all alarming. As business conditions improve, which it is hoped will soon occur, the number may be gradually reduced to a satisfactory level.

(Presented before the California Section meeting, October 26, 1934.)

THE LIVES OF WATER WORKS STRUCTURES

By G. A. Elliott*

(Consulting Engineer, San Francisco, Calif.)

This subject is more or less familiar to all water works men. The operating and construction men encounter it when additions to plant are in prospect and a decision has to be made involving consideration of relative cost and probable length and reliability of service. It is one of the most important elements in the financial side of the business where the useful life of the various component parts of the plant is expressed in the depreciation reserve, affecting the rate which must be charged for service as well as the value of the plant at any given time. The financial aspect of the subject while equally important to both publicly and privately owned utilities receives more attention and discussion in the case of the privately owned companies because depreciation is one of the controversial factors in the fixing of rates, and in determining values either for a rate base or for sale purposes.

The expression of useful lives of structures in financial terms has developed several accounting methods for recognizing depreciation. All are based on assumed lives presumably derived from experience. The more prominent, because most used, are the so-called straight line method which presupposes that the structure in question deteriorates at a regular annual rate requiring setting aside the same percentage of cost each year which will at the end of the assumed life period, without interest, equal the original expenditure; the sinking fund plan which provides for the annual setting aside of a sum of money which at a predetermined rate of compound interest will amount to the original cost at the end of the life period. Obviously either method will give the same result if carried through to completion, and from an accounting point of view both would be equally satisfactory for a plant in continuous ownership. It should be definitely recognized however that at any given time during the life of the structure, the deduction of such accrued depreciation funds from either original cost or present reproduction cost, does not necessarily

* Deceased.

give the present value, as such procedure entirely ignores the actual condition of the structure. The ridiculous result of using theoretical depreciation in determining actual value may be illustrated by taking as an example a structure with a fifty year life and applying first straight line and second a five per cent sinking fund method at an expired life of thirty years. Under the first method the present value is 40 per cent of the original value and under the second 69 per cent.

Useful lives of all utility structures are limited in two ways, by obsolescence and by actual deterioration of the structure to the point where it no longer serves its purpose. Obsolescence is encountered less in water works structures than in almost all other utilities and is practically a negligible factor in affecting useful life. Structures are said to be obsolete when although they may be in operating condition their purpose can be better served by a more modern substitute or by the use of a different method.

Deterioration due to age, wear, weathering, chemical and electrolytic action are the factors principally affecting water works structures. Some materials are inherently long lived such as the familiar cast iron pipe, concrete and earth dams, slow moving pumps and so forth. Others may be made so by proper protection against the attack of destructive elements, such as iron and steel when dipped or painted and given proper maintenance. Climatic conditions due to geographical location is one of the most important factors in determining the length of useful service. The effect of freezing temperatures in the north as contrasted with more equable climates is familiar to most of you and is well illustrated by the difference in character of construction practiced in the New England states and along the Pacific Coast. In the last few years it has been recognized that wood structures are increasingly subject to destruction by termites in a large area of the United States. No native woods are entirely free from the attacks of these insects and because the first visual warning of their presence is sometimes the collapse of the structure, frequent thorough inspections should be made of wooden structures close to or in contact with the ground. Thorough impregnation with creosote is effective as a protector against the action of termites. In general as the result of the combined experience for many years in water utilities located all over the United States, certain average lives have been observed for the various classes of materials, machinery and equipment used in water supply. While

these figures should be modified by local conditions and character of material, some of them are recapitulated here:

	life, in years
Cast iron	100
Steel pipe	25–40
Meters	20–30
Galvanized pipe	20–30
Steam pumps	25–35
Pine flumes	20–25
Redwood flumes	30–40

Maintenance is of course effective in prolonging the service period of structures, especially those that are above ground. In the upkeep of all utility structures, following the advice in the old adage that "a stitch in time saves nine" will return real dividends in good service as well as financially. Regular periodic inspection and constant minor upkeep will increase the time of use. Trueing up shafts and renewing bearings on machinery will often prevent extension of wear to other parts of the apparatus which require major operations to again put them in shape. Replacement of a few boards in a flume will often prevent leaks that are effective in causing rot in a large part of the entire structure. Frequent painting of exposed iron and wood structures and the replacement of small defective parts will postpone the spread of deterioration to the entire unit. Distributing pipe systems are subjected to attack by electrolysis and the cost of frequent surveys followed by remedial measures is not too great a price to pay for the protection of a costly pipe system, to say nothing of saving the cost of the damage that might follow a broken pipe. So prevalent is electrolysis that of late years many utilities are recognizing the situation by installation of insulating joints in both main and service pipes as well as by bonding where such treatment appears to be advantageous.

It is said that exceptions prove the rule and in the case of the ordinary assumed lives of water works structures it may be of interest to cite specific instances of unusual lives due in some cases to unusual conditions. Only a few such instances come within the knowledge of an individual and those mentioned are the result of inquiry among some of the water works men. Cast iron pipe being one of the principal constituents of any water system, there is probably a greater knowledge of its inherently long life than that of any other material. The classic example is of course the French pipe that is so often men-

tioned, where an apparently well authenticated record exists of pipe laid in 1666 which is still in use. Numerous examples of cast iron mains in continuous use for 75 years are recorded in the experience of American cities. A record of pipe laid in Brooklyn, N. Y., shows that 175,000 feet of 6-inch, 103,000 feet of 8-inch and 55,000 feet of 12-inch cast iron laid before 1859 is still in use. Alexandria, Va. has cast iron pipe in the distributing system that was laid in 1850. Portland, Ore. is still using 6- and 8-inch pipe laid over 65 years ago. There is cast iron pipe in the San Francisco distributing system that was brought around Cape Horn from Scotland in 1856 and laid in that year. Deterioration of carrying capacity due to incrustation on the interior of unlined cast iron pipe sometimes presents itself as a functional form of depreciation. This can be remedied by periodical cleaning. Some years ago tests made on a series of 8-inch cast iron pipes indicated reduction in carrying capacity with age at velocities of 3 feet per second tabulated as shown in table 1. Discrepancies in progressive decline can probably be best explained by the fact that the chemical constituents of the water varied as some of it originated in a surface source, some in an underground source, and some was a mixture of the two.

Jointing materials used in the laying of cast iron pipe do not usually affect the life of the pipe itself, but defective joints will greatly increase the cost of maintenance. In recent years two instances of trouble due to joint material have come to notice, causing in one case only loss of water and in the second actual destruction of the line due to cracking of a few feet of each length of pipe beginning at the bell. The second line had a life of about four years. In the first case bacteria originating in the water attacked one of the elements in the joint compound causing excessive leakage. In the second case chemical action apparently caused the joint material to expand thus breaking the pipe.

Rivetted pipe has been used for many years especially for transporting water from source to distributing reservoirs. The early pipes of this character were generally made of wrought iron. Many instances of unusually long periods of service for pipe of this character are found in western water works experience. There is in use today in the San Francisco system 40,000 feet of 12 gage 30-inch rivetted iron pipe laid in 1870 now 64 years old, and 40,000 feet of 44-inch 6 gage pipe 49 years old. In the East Bay area a 37½-inch by $\frac{3}{16}$-inch plate pipe has been supporting a pressure of 80 pounds for the past

49 years and is still in good condition. In Oregon a 30-inch rivetted pipe 51 years old is still in use. In Southern California there is a 30-inch 8 gage pipe 47 years old operating under a head of 100 to 200 feet. In the same vicinity there is a 16 gage 13-inch line 45 years old operating under 20 pounds pressure. As opposed to these instances of long lives there is a record of a 14 gage pipe lasting only 11 years due apparently to an acid soil condition. Another case of accelerated depreciation is given by a line laid in 1926 from which

TABLE 1

AGE	WILLIAMS AND HAZEN, "C"	LOSS
years		*percent*
1 month	108	0
6 months	102	5
1½	87	19
4½	93	14
10	86	20
20	96	11
30	64	41
40	66	38
51	54	50

TABLE 2

DIAMETER	AGE	WILLIAMS AND HAZEN, "C"	LOSS
inches	*years*		*percent*
30	4	114.0	
44	13	113.5	
36	23	109.7	3½
44	27	98.0	14
30	45	91.5	20

the coating has disappeared and the metal is badly pitted and corroded. Due to the slightly alkaline character of the water supply of most California cities prior to the advent of the mountain supplies, the interior of most well dipped pipe has suffered scarcely any deterioration. In practically all cases corrosion and pitting begins on the exterior surface particularly in intensely acid or alkaline soils.

A few records of variation in carrying capacity with age in rivetted pipes may be of interest and are tabulated as shown in table 2.

An instance of a 20-inch continuous wood stave pipe laid in 1908

and estimated to be good for another 10 or 15 years is found in Southern California.

Submarine pipes while not uncommon are not a large part of any system and are always of interest to water works men. The early submarines laid in California in 1887 and 1888 are still in use and so far as condition is concerned they appear to still be in their early youth. To supply Coronado 4,200 feet of cast iron pipe were laid under the bay in 1887 and is still in use after 47 years of exposure to salt water. In 1888 12,000 feet of 16-inch lap welded tubing galvanized and dipped in a mixture of asphaltum and coal tar was laid in two lines across San Francisco Bay at Dumbarton. After 46 years of continuous use the exterior of about one-half of the line which can be inspected during the annual lowest tide is in good condition with the dip still adhering to the galvanized surface.

Fortunately for the water works business progress in reciprocating pump design has not rendered some of the older machines obsolete. One sometimes hears of old Cornish pumps which are still used, but the record breaker so far as the writer's information goes is the case of the reciprocating pump purchased in 1851 and installed in Alexandria, Virginia. It has a 100-inch stroke and is operated by an overshot water wheel. It is still used when there is sufficient water to operate the wheel. It is interesting to note that this pump is located in an old mill building constructed in 1800 and reported to be in excellent condition. The ditch through which water is brought to this station is also 134 years old. Coming back to the Pacific Coast there is the example of the cross compound duplex steam operated units at the Belmont Pumping Station of the San Francisco Water Department. They were installed in 1889, 45 years ago, and have given continuous efficient service since that date. Portland, Ore. is still operating two reciprocating pumps 40 years old.

A few examples of miscellaneous units may be of interest. San Jose Water Works is now replacing a redwood flume which has been in continuous use since 1888 or 46 years. There is still in use in the San Francisco Bay area a small redwood flume 55 years old, and a larger redwood box flume used for occasional waste discharge in winter which is 59 years old. A 250,000 gallon rivetted iron tank located on the top of a San Francisco hill where it is exposed to sea fog is still in excellent condition after 49 years of service.

If there is any lesson to be learned from the foregoing discussion it is that having selected the best material that can be afforded, con-

stant vigilance in maintenance and upkeep will result in maximum life of structures and will be reflected in decreased cost of service. A full knowledge of all the conditions and the selection of the proper plant to meet these conditions whether it be pipe, machinery or any of the numerous other appurtenances of a water works, is necessary for efficient operation and service in any utility. Last but not least the present condition of the component parts of any utility plant cannot be determined by the application of a percentage derived from the relation of expired life to assumed total life based on general records of experience. Such figures have only a very general relation to an actual case and should be modified by the actual conditions found by thorough inspection of the plant.

(*Presented before the California Section meeting, October 25, 1934.*)

EXPERIENCE WITH FLUORINE IN WATER

By Jane H. Rider

(*Director of State Laboratories, University of Arizona, Tucson, Ariz.*)

This is a brief review of the data which have been published relative to fluorine in potable water supplies and its physiological effects upon the consumer. The study of the occurrence of this element in water is not one of the fads or fancies that occasionally intrigue research chemists, but a serious investigation brought about by the real need of afflicted communites. In 1915 the residents of Mammouth appealed to the University of Arizona to determine the cause of a discoloration appearing on the teeth of the children living in the village. Some time later St. David called a similar condition to our attention. The dentists recognized a serious dental defect and suggested iron or manganese in the water as the possible cause of the stain. The chemical analysis of the water did not confirm this suspicion nor could it be accounted for as lack of calcium. The condition continued a mystery and the children to have defective teeth.

When articles on vitamins began to appear that was a fine alibi for a water laboratory, undoubtedly it was a nutritional deficiency. This, however, did not satisfy the dentists. In 1918, F. S. McKay (1) called attention to the occurrence of mottled enamel in areas using artesian water; in 1926 (2) and again in 1928 (3), he implicated water as the cause of this disfiguration of the teeth. H. V. Churchill (4) in April, 1931, called attention to fluorine in waters from locations where mottled enamel occurred, but did not assign this as the definite cause for the dental condition.

In June, 1930, Smith, Lantz, and Smith (5), started a project at the University of Arizona to investigate the occurrence of mottled enamel in several small communities in the southern part of the State and to determine, if possible, the etiological factors concerned in producing this condition. Several members of the Tucson Dental Association, headed by Dr. A. E. Bard, cooperated in examining the teeth of every child in the schools at St. David and recording the findings on a special individual chart. Later the history of each

family was obtained. A careful study of these data indicated that everyone born and reared in the community or those who came there very young showed mottling of all the permanent teeth. The individuals who moved to the locality after the permanent teeth had developed did not show any characteristics of mottling. There were apparently no hereditary tendencies and the local environmental condition was only effective when the enamel of the permanent teeth was being formed. These findings preclude a dietary deficiency, because the food habits of families would not vary radically in moving from one farming locality to another.

Factual evidence compiled seemed to bear out McKay's supposition that water supplies were involved. It did not necessarily seem to be artesian wells, as he had implied, since families using shallow dug wells were also affected. The ordinary chemical analysis did not indicate any probable causative agent. Current literature supplied information concerning the effects of fluorine compounds on the development of the enamel of rats' teeth when these compounds were added to the normal diet. The dental conditions cited in these reports clearly resembled the appearance of the teeth of the rats being fed concentrated water from St. David. Qualitative analysis of the water showed the presence of fluorine, and it became necessary to find an accurate method for the quantitative analysis. (The bibliography appended to this paper lists the various methods which have been developed recently to determine the percentage of fluorine in water.) The Reynolds, Ross, and Jacobs (6) method modified by H. V. Smith showed the well waters in this district contained from 3.8 to 7.1 p.p.m. fluorine while the water from non-endemic areas contained only 0.0 to 0.3 p.p.m. This seemed to advance conclusive proof that mottled enamel was a defect caused by a definite concentration of fluorine in drinking water.

A brief description of the dental defect called mottled enamel may emphasize its seriousness. It is characterized by dull chalky white patches unevenly distributed over the surface of the tooth. In aggravated cases the whole surface of the tooth may present this unglazed appearance, the teeth losing their natural translucency. During this early paper white stage the imperfection is not especially noticed by casual observers. However, as the child grows up, the enamel frequently becomes corroded and a stain, commonly called brown stain, appears on the teeth, varying in intensity from yellow to almost black. It is unfortunately especially marked on the exterior

surfaces of the upper teeth so it is rather conspicuous. The nature of the structural weakness of these teeth make it difficult for dentists to put in satisfactory fillings. There is a social aspect of this problem that cannot be ignored. Young people, their teeth marred by mottled enamel, find it difficult to secure high types of positions. The economic cost of dental care of these defective teeth is also a serious matter to the families of these children. The dental group at Tucson stated that a conservative estimate of dental costs per person afflicted with mottled enamel, who makes the attempt to preserve his teeth, will be approximately a thousand dollars. This will carry them to an average age of thirty years when they are very apt to lose all their teeth and must resort to artificial substitutes. Aside from any academic considerations there is a very definite financial loss to the citizens of those communities which have water supplies with a high fluorine content.

In considering the quantity of fluorine in a water that will affect the enamel when the permanent teeth are being formed the method of determining the amount of fluorine is important. If the Fairchild (7) method is used the toxic level will be from 2.0 to 2.7 p.p.m.; any water having less than 2.0 p.p.m. will be safe for young children. The methods of Foster (8), Willard and Winters (9), Sanchis (10) modification of the Thompson and Taylor or Steiger (11) indicates that from 0.9 to 1.0 p.p.m. is the limit of toxicity and water to be safe for all children should have less than 0.7 p.p.m. In determining the fluorine toxicity of a water supply the water department must not depend entirely upon the chemical analysis. It is far more important to have a dental survey made of the school children. Such an investigation should be carried on by trained personnel familiar with mottled enamel and should include enough of the personal history of the child to indicate where they were living when the permanent teeth were being formed. Such a survey will show more clearly than any chemical analysis of the water whether or not the teeth of the children are being affected, and if such dental defects are occurring in a considerable portion of the children, the municipal governing body should consider seriously alleviating this condition.

The various experimental methods that have been proposed to remove fluorine from the water have not been entirely successful. The quantity of fluorine occurring in water is so minute and its toxicity so great that it will be difficult to find a substance that will reduce the quantity below the toxic level. The writer has coöperated

to a small extent with H. V. Smith in his fluorine removal studies. We have tried to check the methods suggested by A. Carnot (12), Boruff (13), and McKee and Johnson (14). The economic aspects of the cost of treatment will make these methods of doubtful practical value even if the percentage of fluorine is reduced below the toxic level. Our experiments made with 37 gallon batches of water fortified with sodium fluoride were not encouraging either from the point of view of removal or economy.

In those states where mottled enamel occurs fluorine in water has become one of the major water works problems. H. Trendley Dean (15), Dental Surgeon, U. S. P. H. S., lists 97 localities where mottled enamel has been definitely demonstrated, 28 areas referred to in literature where no confirmatory surveys have been reported, and 70 areas which have been reported by questionnaires not confirmed by extensive surveys. When the high cost for dental care to the citizens of these communities is considered, it may prove less expensive to establish treatment or secure a new source of supply. There is also the possibility of families, with children of susceptible age, using distilled water exclusively for drinking and culinary purposes. Certainly the fluorine content of all new water supplies or additions to existing supplies should be determined. In this respect dilution with waters with zero or low fluorine content may furnish a factor of safety. It may also be possible to extend municipal supplies to adjoining rural districts where private well supplies have toxic amounts of fluorine.

There is a possibility that fluorine in foods may intensify the physiological effect of the fluorine in the water during the period in which the permanent teeth are being formed. Such an effect might be increased further if the vegetables were grown in soil with an increased percent of fluorine or irrigated with water containing fluorine.

Hart, Phillips, and Bohstedt (16) have suggested that when soils are fertilized with fluorine carrying phosphate, the fluorine content of the ground water may be increased. Willis (17) at Colorado Springs indicates that small amounts of fluorine may be dissolved from the rocks by water flowing over them. An attempt should be made to trace the origin of fluorine contaminations. It will be necessary for the geologists and water chemists to coöperate closely in making such a study.

A standard method for fluorine determination should be adopted

since the results of the various workers should be readily comparable. So small a quantity of this substance in a water supply has such an adverse affect upon the young children of a community that accurate analysis becomes especially essential. Further, water works officials must have a clear understanding of the amount of fluorine that will cause mottled enamel.

In conclusion there are several essential facts that water works men should keep in mind in considering the subject of fluorine in water supplies:

1. Water containing from 0.9 to 1.0 p.p.m. fluorine will cause a dental defect, known as mottled enamel, if consumed by children of susceptible age. For children particularly sensitive it should be below 0.7 p.p.m.

2. This defect may cost an afflicted individual a large sum for dental care as well as a certain loss of personal prestige.

3. A careful dental survey of all children in the schools must be made in addition to the chemical analysis before a water supply is condemned.

4. When a water contains a toxic amount of fluorine there is the question, should the individual citizen continue to pay large dentist bills or should the community secure a new water supply?

5. To date no satisfactory, economical methods of removing fluorine have been developed.

6. Other sources of fluorine intake into the system should be investigated and further attempts made to determine the origin of fluorine contamination of water.

REFERENCES

(1) McKay, F. S.: Progress of the year in the investigation of mottled enamel with special reference to its association to artesian water.—Jour. Amer. Dent. Assoc., 5: 727–750, 1918.

(2) McKay, F. S.: Water supplies charged with disfiguring the teeth. Water Works Engineering, 79, 1926.

(3) McKay, F. S.: Evidence that mottled enamel prevalent in Italy is caused by the water supply. Water Works Engineering, 1928, p. 1037.

(4) Churchill, H. V.: Occurrence of fluorides in some waters of the U. S.—Jour. Ind. & Eng. Chem., 23: 9, pp. 996–998, 1931.

(5) Smith, M. C., Lantz, E. L., and Smith, H. V.: The cause of mottled enamel, a defect of human teeth. Univ. of Arizona, Agr. Exp. Sta. Tech., Bul. No. 32, 1931.

(6) Reynolds, P. S., Ross, W. H., and Jacob, L. D.: Volatilization method for the determination of fluorine with special reference to the analysis of Rock Phosphate. Jour. Asso. Agri. Chem., 11: 225, 1928.

(7) FAIRCHILD, JOHN G.: The volumetric determination of fluorine by the use of ferric chloride. Jour. Wash. Acad. of Sciences, vol. 20.

(8) FOSTER, MARGARET D.: Colorimetric determination of fluorides in water using Ferric chloride. Ind. & Eng. Chem. Anal. Ed., 5: pp. 234–236, 1933.

(9) WILLARD, H. H., AND WINTERS, O. B.: Volumetric method for determination of fluorine. Ind. & Eng. Chem., Anal. Ed., 5: pp. 7–10, 1933.

(10) SANCHIS, J. M.: Determination of fluorides in natural waters. Ind. & Eng. Chem., Anal. Ed., 6: pp. 134–135, 1934.

(11) STEIGER, G.: Colorimetric determination of fluorine. Jour. Am. Chem. Soc., 30: pp. 219, 1908.

(12) CARNOT, A.: Recherches sur la composition generales et la teneur en fluor des os modernes et des os fossiles des differents ages. Ann. Mines (9), 3: 155–195, 1893.

(13) BORUFF, C. S.: Removal of fluorides from drinking water. Jour. Ind. & Eng. Chem., 26: 1, p. 69, 1934.

(14) MCKEE, R. H., AND JOHNSTON, W. S.: Removal of Fluorides from drinking water. Ind. & Eng. Chem., 26: pp. 849–51, 1934.

(15) DEAN, H. TRENDLEY, SEBRELL, W. H., BREAUX, R. P., ELVOVE, E.: Effect of various amounts of sodium fluoride on the teeth of white rats. Public Health Reports, 49: 37, September 14, 1934.

(16) HART, E. B., PHILLIPS, PAUL H. AND BOHSTEDT, G.: Relation of soil fertilization with superphosphates and Rock Phosphate to fluorine content of plants and drainage waters. Am. Jour. Pub. Health, 24: 9, September, 1934.

(17) WILLIS, WILLET: The source of the fluorine in some water supplies. Bulletin of the Colorado State Dental Asso., pp. 39–44, 1934.

SMITH, H. V. AND MARGARET CAMMACK: Mottled Enamel in Arizona and its correlation with the concentration of fluorides in water supplies. Univ. of Ariz., Agri. Exp. Sta. Tech. Bul. No. 43, 1932.

GOTTLIEB, SELMA: Fluorides in water and their relation to mottled enamel. S. W. Water Works, Jour., October, 1934.

MCKAY, F. S.: Brief statement of the case against fluorine in water as the cause of mottled enamel. Production stopped at Oakley, Idaho by a change in water supply. Jour. Dental Research, 13: 133, 1933.

DISCUSSION

CARL WILSON (*Los Angeles, Calif.*): Miss Rider has given us an excellent conception of the importance of fluorine in domestic water supplies, and the water works man will be glad to have the bibliography which she has appended to her paper, for it covers all the literature on the subject which he is likely to require.

Troublesome amounts of fluorine seem largely confined to well and spring waters, and hence affect only smaller supplies. Large municipal supplies will probably not often be concerned with this element. Unfortunately the smaller volumes are more difficult and expensive

to treat, and it seems doubtful that fluorine removal will prove practical. Some success has been reported with the use of activated carbon as an absorbent, but it is necessary to drop the hydrogen ion concentration to a pH of 3.0 to make the process efficient. Subsequently, of course, it would be imperative to raise the pH to perhaps 8.0 before use. The trouble and expense might be justified in some extreme cases, but in general the real solution of the problem will be the substitution of a new supply which is fluorine-free.

There is some reason for believing that objectionable amounts of fluorine are found only in waters which come in contact with volcanic emanations or volcanic sublimations. If this should be the case it is natural to suppose such waters would also contain mineral amounts of boron, and it would be worth while to look for boron in all fluorine-bearing waters, or, conversely, to look for fluorine in all high boron waters. Should the presence of these elements be connected with volcanic phenomena that fact might aid us in the search for suitable waters.

Miss Rider has displayed a keen appreciation of the practical side of the problem in calling attention to the great superiority of clinical evidence over analytical results in determining the toxicity of a given water. The quantitative determination of this element in water is fraught with great difficulties, for a number of elements and radicles interfere with the different methods so far available. Undoubtedly these technical difficulties will be overcome in time, but at present no method is known which is sufficiently accurate and dependable to permit readings to a tenth part of a million. In this respect I think Mr. Sanchis is to be congratulated upon having so selected his standards as to report fluorine as absent, less than 1, between 1 and 2, and over 2 p.p.m. This scale gives all that the practical water works man needs to know, and is as refined as present methods permit. In critical cases it is possible a long series of determinations, through application of the law of averages, may be made to yield closer figures, but it is doubtful if such trouble and expense would often be justifiable when clinical evidence among consumers of the water is available.

A case in point is a recent report that the Colorado River water contained so much fluorine as to make it undesirable for domestic use. The allegation died a natural death for clinical evidence furnished by thousands of persons who had used the water exclusively for all domestic purposes over a long period of years proved conclu-

sively that mottled enamel has never been caused by Colorado River water.

Miss Rider has wisely stressed the need of a standard technic for fluorine determinations, for it is common experience that the same worker using different methods on one water will secure widely different results. Some one method should be selected and used in all laboratories, and the determination should be made on all waters intended for domestic use. Pending action by the Committee on Standard Methods, it might be well for the directors of state, county and municipal laboratories in California and Arizona through mutual agreement, to select a tentative method in order to sooner produce a mass of comparable data on waters in these states.

G. L. CHENEY (*Chief Chemist, Smith-Emery Company, Los Angeles, Calif.*): The paper presented is very interesting and indicates that the question of fluorine in water is one which should be given careful consideration both from a chemical and a physiological point of view. As stated in the paper, although stained or mottled condition of teeth has been noted for many years, it has only been in the last three or four years that it has been definitely connected with fluorine content of waters, and intensive study given the matter by various groups.

It appears that the problem has three phases. First, dental examinations to determine where this condition exists and its extent. Second, chemical methods which definitely show the amount of fluorine present (as suggested in the paper, a method should be standardized so that results reported from various workers are comparable). Third, the development of processes whereby the fluorine content of water supplies may be reduced below the toxic limit.

Considering these three phases I should like to ask the following questions in an endeavor to broaden the scope of the question and if possible to get a clear concept of its problems and magnitude.

First is in relation to the number of persons actually or potentially affected in this country. The paper just given refers to certain areas, but I should like to know in the opinion of the writer the general location of these areas, the number of persons affected, and the probable completeness of the present surveys. In this connection what relation has food supplies which contain fluorine in appreciable amounts accumulated from the soil directly or from the irrigating later used. Is the shipment of such foods going to complicate the problem or will areas where high fluorine occurs be quickly deter-

mined and segregated for agricultural purposes. This now brings up the question as to the relative action of organically combined fluorine to the mineral salts. If any one present has any statement to make on this question I think it would be of great interest.

In the second place I should like to ask the opinion of the writer which in her mind is the best method for the general determination of fluorine. We have examined many methods in our laboratory, but at present feel that the method used by Mr. Sanchis gives very satisfactory results.

Thirdly as to the probable removal of fluorine by treatment. If the writer has done any work on this subject, even though the results were not satisfactory, it would be of interest to know just what has been done, as many times work that was not successful produces ideas which may be of later value.

(*Presented before the California Section meeting, October 25, 1934.*)

EFFECT OF METER MODERNIZATION IN GERMANY[1]

By H. W. Niemeyer

(*Indianapolis Water Company, Indianapolis, Ind.*)

It is a fact that one can learn more grammar by the study of a foreign language than through the study of his own speech. By the same reasoning it is entirely possible that the water plant operator can improve his knowledge of water measurement by knowing what his foreign neighbors are doing in the same field. So it has been with a great deal of interest that two articles appearing in German gas and water works journals were read and of which a brief résumé is herein presented.

In Germany, as in this country, there has been considerable effort on the part of the administration to create work for the jobless. Among those businesses where aid was lacking or held up, were the civic works providing gas, water, and electricity, or the amount of financial assistance received was of such small degree that the modernization and improvement of water works could only be carried out in part. It was for this reason that Denkert in his article "Water Meters and Water Waste" attempted to point out means of creating work in the water works field that would at the same time benefit the operator through increased income. This means was the elimination of water waste through the replacement of outmoded meters with ones of modern design.

That water waste does exist, Denkert gives statistics as shown in table I.

In discussing these data he says in part:

> If we consider these figures, we see that not only in the large but especially in the medium sizes and smaller water works there is enormous waste of water, that even today a waste of 30 or more percent is not exceptionable. We know that the waste in the distribution system, if it is in order is scarcely 2 to 3 percent of the output. We know, too, that using sensitive meters the loss

[1] Abstracted from: "Water Meters And Water Waste" by Denkert in Das Gas- und Wasserfach, 78: 4, 1935. "Standardizing Water Meters" by Gotting in Das Gas- und Wasserfach.

appearing through the meters need not be higher than 8 percent so that a total water waste of approximately 10 percent is fitting and understandable. If the distribution system is in order, then evidently the present unjustifiable high waste is on the part of the consumer. The explanation is not hard to find. We know that in Germany the flywheel water meters[2] are chiefly used, that further the flywheel water meters let small amounts of water through up to the point where they begin to indicate correctly with a tolerance of plus or minus 2 percent. Small leaks in the house piping, dripping faucets and small closet flows are not indicated by the flywheel meter so that we do not get an exact measure of total amount of water passing through it. If these wastes are not attended to, they lead to immense quantities.

TABLE 1

PLACE	POPULATION	ANNUAL OUTPUT	SOLD THROUGH METERS	LOSS
		cu. m.	*percent*	*percent*
A	3,387,000	163,721,740	90.5	9.5
B	573,000	30,300,710	67.0	33.0
C	733,000	28,088,657	90.0	10.0
D	617,000	20,754,736	84.5	15.5
E	71,000	2,833,900	64.0	36.0
F	76,000	2,764,027	89.5	10.5
G	76,000	2,665,625	71.0	29.0
H	38,000	1,539,519	70.0	30.0
J	59,000	1,470,290	76.0	24.0
K	27,000	1,400,000	61.0	39.0
L	30,000	795,165	76.5	23.5
M	36,000	793,651	67.0	33.0
N	20,000	790,160	89.5	10.5
O	18,000	480,552	70.5	29.5
P	24,000	457,364	76.0	24.0
Q	12,000	456,534	61.0	39.0

Mr. Denkert in his article presents considerable evidence that the German plant operators have done intensive research work in the field to determine rates of flows at which water is used through meters and the amount of under-registration that occurs in service. Two types of rate of flow recorders used for these studies were illustrated in his publication and these are duplicated here.

Figure 1 details the recording mechanism used on a special displacement type meter which they call a "trouble-shooter." The

[2] The flywheel meter is the "jet type" dependent on water velocity for propulsion.

measuring compartment of this meter is of the annular piston type[3] and presumably very sensitive. It is interesting to note that the measuring chamber as well as the piston is made of hard rubber because it withstands corrosion and experience has shown it to measure accurately unusually long. The recording device consists of a standard register that also actuates a vertical pin over a clock driven roll chart. The trouble-shooter is set in series with the house meter to be tested and the recordings of each observed to secure the actual under-registration of the house meter while a time study is recorded of rates of flows used.

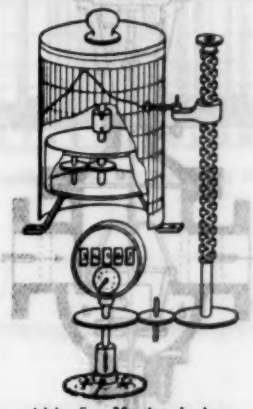


Abb. 5. Mechanischer Antrieb der Schreibvorrichtung eines Schadenszählers (schematisch).

FIG. 1

Figure 2 illustrates a second type of rate of flow recorder known as a "Swim-meter" and figure 3 the type chart that is obtained with its use. Note that float "C" set in a venturi opening rises according to the rate of flow passing through the meter. This movement actuates the chart pin so that rate of flow is directly indicated.

Through the use of this equipment the Germans have proven quite conclusively that their older fly-wheel types of meters were not accurate because they were not sensitive to small rates of flow. They have found that by replacing these older meters with modern ones having a high degree of accuracy, they can either eliminate unaccounted for water or reduce it to a reasonable percent.

[3] The annular piston meter is the eccentric piston or displacement type.

Figure 4 is a reproduction of accuracy curves for various models of German 5 cm. meters. Curve (a) for a 1906 flywheel meter shows it to be registering inaccurately for flows less than 100 liters per hour or 0.44 gallon per minute. The 1928 flywheel meter curve (b) shows

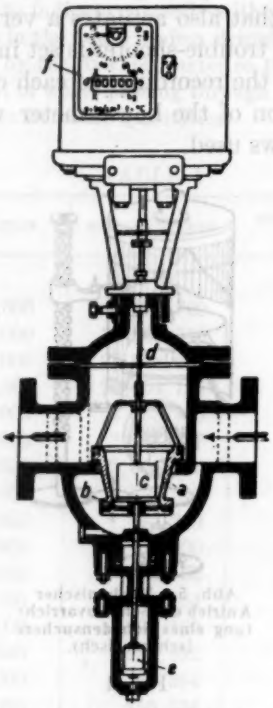


Abb. 6. Schwimmerwassermesser, Schnitt.

a = MeBdüse,
b = Schwimmerteller,
c = Belastungsgewicht,
d = Verbindungsstange,
e = Dämpfung,
f = Zählwerk.

FIG. 2

to register correctly at 50 liters per hour or 0.22 gallon per minute. Curve (c) shows the accuracy of a 1930 annular piston type meter, it being correct for flows down to 20 liters per hour or 0.088 gallon per minute.

The improvement in these water meters has been brought about gradually through several ways. The complete application of im-

proved machinery in the meter factories has been a very deciding factor to this end. The use of non-corrosive materials has greatly improved the meters. Probably the most important change in the design of the flywheel meter has been the weight reduction of the

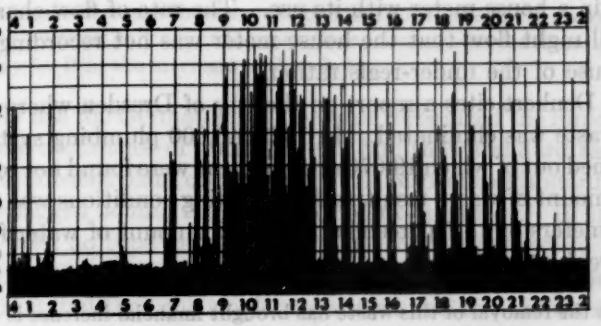


Abb. 7. **Tagesdiagramm der Belastung einer Rohrleitung.**

FIG. 3

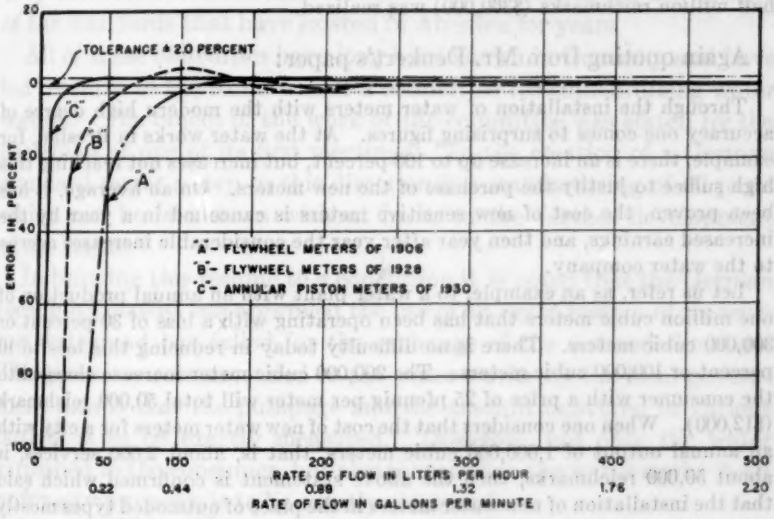


FIG. 4. ACCURACY CURVES FOR GERMANY 5 CM. METERS

flywheel itself which has influenced appreciably the initial accuracy of the meter. The submerged weight of the 1906 model flywheel was 11.2 as compared with 5.7 grams for the 1928 model.

Together with the replacement of outmoded meters there has been

considerable attention directed towards their proper sizing. This without doubt has played an important part in the reduction of wasted water.

An example was cited to typify the results of studies with the "trouble-shooter" meter. An under-registration of 13.1 percent was found in a house meter with its use. The rate of flow chart showed a small night flow that the house meter was not recording and was the cause of the under-registration.

Mr. Denkert cites a report on the city of Dresden where considerable waste was eliminated. In 1928, 46,000 plumbing systems were examined out of which 19,670 or 44 percent were found not to be tight. The amount of waste resulting from leaking conditions was 6,350,000 cubic meters or 18.7 percent of the total amount of water furnished. Mr. Denkert states:

> That the removal of this waste has brought financial increase is seen in the fact that waste in Dresden was reduced from 29.9 to 8.9 percent or 21 percent. Taking 1932 output of 29,000,000 cubic meters as a basis and a price of 24 pfennig (about 6¼ cents) per cubic meter, a yearly increase of about one and a half million reichmarks ($360,000) was realized.

Again quoting from Mr. Denkert's paper:

> Through the installation of water meters with the modern high degree of accuracy one comes to surprising figures. At the water works in Breslau, for example, there is an increase up to 100 percent, but increases not reaching this high suffice to justify the purchase of the new meters. On an average, it has been proven, the cost of new sensitive meters is cancelled in a year by the increased earnings, and then year after year the considerable increases accrue to the water company.
>
> Let us refer, as an example, tc a water plant with an annual production of one million cubic meters that has been operating with a loss of 30 percent or 300,000 cubic meters. There is no difficulty today in reducing this loss to 10 percent or 100,000 cubic meters. The 200,000 cubic meter increase charged to the consumer with a price of 25 pfennig per meter will total 50,000 reichmark ($12,000). When one considers that the cost of new water meters for a city with an annual output of 1,000,000 cubic meters, that is, about 2,000 services, is about 50,000 reichmarks, then the above statement is confirmed which said that the installation of new water meters in the place of outmoded types mostly pays for itself in a single year.

Mr. Gotting in his article "Standardizing Water Meters" gives the following example:

> In the Berlin water works in the last years the amounts not caught by water meters have been reduced from 9 to less than 6 percent. Such improvements which at the same time creates work can be obtained, first, by the use of new

meters with a good degree of accuracy in place of outmoded meters; second, by the prompt exchange of installed meters; third, by conscientious repairs and careful tests; fourth through the selection of meters of the proper size. The money spent on this work brings good return. The Berlin water works, with a yearly output of 160 million cubic meters, by decreasing meter under-registration, increased their yearly revenue 400,000 reichmarks ($96,000). I have touched this point because there are decided increases possible in many places, and because we must endeavour to teach the consumer who continually wastes water to pay for it in the interests of the whole.

Not only has the German plant operator recognized the importance of low flow accuracies of their meters, but that standardization is essential to efficient plant operation. With respect to standardization they have lagged behind the American plant operators. In the past there has been no type meter in Germany that was standard among the manufacturers as to length, size, capacity, connections, or registers. It is stated that one single meter firm had to produce a single size meter in 65 different sorts, in order to meet the various specifications of plant operators. One has only to imagine the difficulties that they encounter in their meter maintenance to realize the advantages of the standards that have existed in America for years.

All of these difficulties have long been known in Germany and have led recently to the formation of a committee for standardizing water meters. The object of the work of the committee was stated by the chairman at one of its first meetings: "In the creation of a German standard meter a step in this direction is a standardizing of all parts which can be laid down without harm to the further development of the meter."

In bringing this résumé to a conclusion it is noted that the German local government has approved the use of an exact system of measuring water and has called for the exchange of the outmoded meters. The measures recommended to the water works by the local government means that the pumpage and the amount delivered to various outlets is to be exactly measured. Therefore, not only the water furnished to the consumer, but also that used in the water works itself or for city purposes is to be exactly measured.

The American plant operators have been so busy fully metering all services that but little attention has been paid to the accuracy limitations of their meters and resultant losses from wasted water. We have but to look at our records of unaccounted for water to realize that we are doing no better than our foreign neighbors and are allowing much waste to go by unheeded.

Attention is called to the fact that standardized and precision

tooling operations have only been introduced in our meter factories in the last decade. Also in this period great advances have been made in the use of non-corrosive metals. A reduction in friction has been brought about through the use of oil enclosed gear trains. So that the 1935 American meters are much improved as to accuracy over early makes, and we too, have possible large savings available through the replacement of outmoded meters. If American plant operators do not care to accept the evidence of Germany's findings, at least they should plan intensive research work to determine the extent of possible savings that can be realized through the use of modern meters, properly sized, accurate, sensitive, and properly maintained.

In remarking that Americans were in advance of Germans as to standardized meters, it was not intended to mean that we had gone as far as we can in establishing standard specifications. On the contrary, it is felt that existing meter specifications can be both altered and expanded to insure the plant operator an accurate measurement of water sales and an accuracy that will endure over a long period of time with a minimum maintenance cost.

MUNICIPAL GROUND WATER SUPPLIES OF ONTARIO

By W. B. Redfern

(Of James, Proctor & Redfern, Ltd., Consulting Engineers, Toronto, Ont., Can.)

A perspective of the municipal ground water supplies of Ontario is shown on the accompanying map of the province. The location and names of the municipalities using ground water supplies entirely or in part are shown on the map. At a glance one sees that most of these municipalities are situated in the south-western peninsula. A few towns in the northern section of the province obtain their supply from wells, as well as a small number in the eastern part of the province.

In Ontario, there are 281 municipalities which have waterworks systems in connection with which 226 have their own source of supply. The other municipalities purchase water. Of the total number, 80 obtain their supply from deep wells; and of the deep-well supplies, 35 are in rock formation, and 41 are from wells terminating in glacial deposits of sand and gravel above the level of the rock. And four municipalities obtain supplies from wells in both rock formations and glacial drift.

In the United States, it has been calculated that there are approximately 11,000 municipalities that have waterworks systems, of which number about two-thirds have ground water supplies. In Ontario, the fraction is considerably less than two-thirds because many municipalities border on the Great Lakes, and, because there are quite a number of smaller lakes and rivers which discharge into the Great Lakes and which afford surface-water supplies.

The importance of ground water supplies is becoming more recognized and appreciated. More intensive study has been made of the quality, quantity, and location of ground water supplies. Especially for the smaller municipalities, supplies from deep wells, where available, are usually reliable and economical, and afford a potable, palatable, and wholesome supply of water for domestic consumers.

The occurrence of ground water supplies in Ontario varies greatly

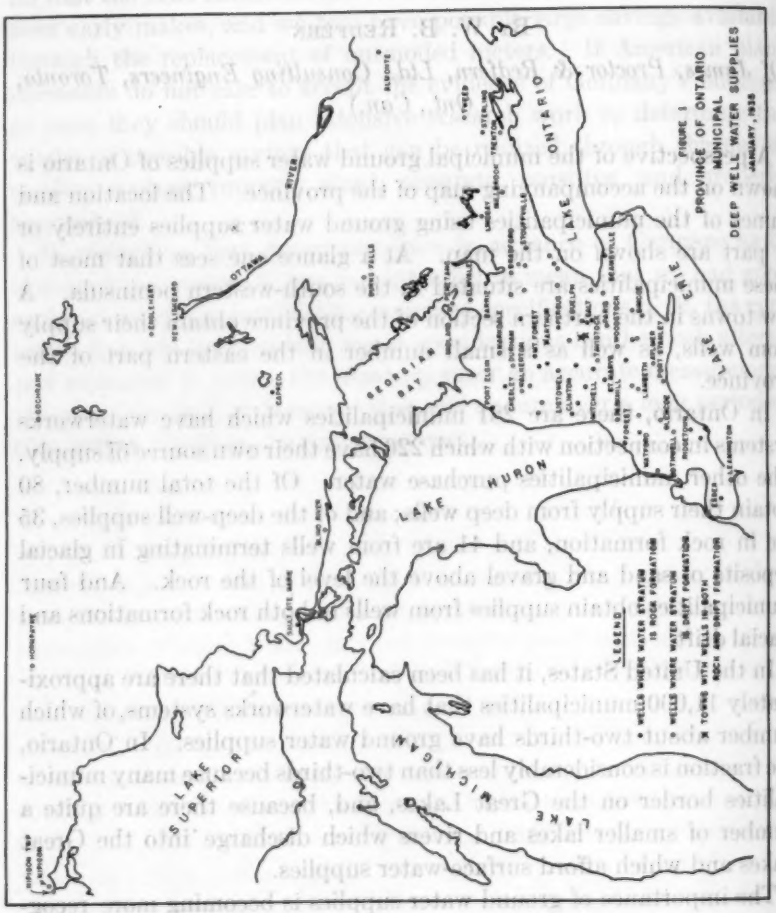

PROVINCE OF ONTARIO
MUNICIPAL
DEEP WELL WATER SUPPLIES
JANUARY, 1938
LEGEND
WELLS WHERE WATER STRATUM IS ROCK FORMATION
WELLS WHERE WATER STRATUM IS DRIFT FORMATION
TOWNS WITH WELLS IN BOTH ROCK AND DRIFT FORMATION
LAKE SUPERIOR
LAKE HURON
LAKE ONTARIO
LAKE ERIE
LAKE MICHIGAN
GEORGIAN BAY
OTTAWA RIVER

FIG. 1

with the complexity of geologic structures, and with the character of the glacial drift occurring in irregular deposits throughout the province. In the north country, there are vast masses of hard igneous rocks, near and at the surface of the ground, that contain little water, and that are difficult to penetrate. In this great area there are also patches of glacial deposits with interbedded strata of sand, gravel, boulders, clay, fine stone dust, and till.

In the south-western peninsula, the igneous rocks are overlaid with limestone and shale of various thicknesses and ages, containing frequently large quantities of water in fissures, cracks, faults, crevices, and solution channels. The water in these secondary interstitial openings is usually highly mineralized. Above the limestones are glacial deposits containing varying quantities of water depending largely upon the thickness, extent and character of the sand and gravel beds. In the eastern part of the province, the underlying limestone and shale formations are not good aquifers, and, as a result, few supplies are obtained from deep wells. In this section, there are a few wells with casings finished in glacial deposits.

EXAMPLES OF WELL SUPPLIES

It might be interesting to describe in detail some of the deep wells in the province used for municipal supplies.

These municipalities include Capreol and Sault Ste. Marie in the north; Milverton, Seaforth and Hespeler in the south-west; Newmarket, Bradford, Barrie, Penetanguishene and Ancaster in the south-centre; and Almonte in the east. The description of these deep well supplies is in the nature of a cross-section of the ground water municipal supplies of the province.

In *Capreol*, there are two wells of the gravel wall type installed immediately outside the east boundary of the town in a shallow glacial deposit of gravel and fine sand. The wells are 300 feet apart, and are 42 feet and 44 feet deep respectively. They consist of an outer casing 26 inches in diameter, and an inner casing 16 inches in diameter. The first well was put down by the rotary clay-seal process, and the second well was bailed in. They have a combined capacity of 398 Imperial gallons per minute. In well 1, the static level of water is 9 feet from the surface. When pumped at the rate of 218 g.p.m., the total drawdown was 24 feet, or the specific capacity was 9 gallons per minute per foot of drawdown. The water is soft and is of a suitable quality for a municipal supply. In separating the water

from the fine sand formation, it appeared that the only effective way to accomplish this result was by the construction of a gravel wall well of ample dimensions to make certain of the installation in a positive manner of a complete column of pea gravel artificially placed around the screen from top to bottom, having the screen as the core. The annular space between the casings is 5 inches. Both casings are installed to the surface of the ground, and the annular space is kept filled with pea gravel. The deep well pump is installed in the 16-inch inner casing, and pumps water against standpipe pressure, automatically controlled. At the time of installation it was a matter of some surprise to the writer that the quantity of 398 g.p.m., or well over 500,000 g.p.d., could be developed from this formation of comparatively shallow depth above the rock level.

In an installation of this type, where the municipality is dependent entirely on a deep well supply, it is advisable to have two wells completely equipped as alternative sources of supply.

In Sault Ste. Marie, a city of 23,000 population, the water is supplied to the western section from deep wells originally installed to supply the Town of Steelton, now a part of the city. For 20 years these wells have supplied approximately 700,000 g.p.d., as an artesian flow from the sandstone formation, 70 feet below the surface without any apparent diminution in supply or lowering of ground water levels. Horizontal centrifugal pumps discharge the water into the distribution system.

Last year it was decided to put down a large well consisting of a 24-inch casing to the rock level, and a 20-inch hole in the Lake Superior sandstone to a depth of 20 feet. During a continuous pumping test of seventeen days, the maximum amount of water pumped, at stabilized conditions, was at the rate of 2,500 g.p.m. The specific capacity of the well was 27.5 g.p.m., per foot of drawdown. This well has a remarkable capacity. The water apparently comes from the original interstitial spaces between the sand grains now consolidated and cemented into sandstone.

A 6-inch test hole was put down to a depth of 48 feet below the bottom of the large well, but no more water-bearing horizons were encountered. This Lake Superior sandstone is undoubtedly the best aquifer of any rock formation in the Province of Ontario. At its maximum capacity, this well could furnish 10 percent of all the well water obtained in the province at present for municipal waterworks systems. The water is soft, having a total hardness of only 53 p.p.m. and is clear, cold, and wholesome, and is ideal for a municipal supply.

In the *Village of Milverton*, a rock well, 372 feet deep, supplies sufficient water for the municipality. Rock was encountered at a depth of 140 feet. The static level of water in the well is 32 feet from the surface. Fifteen years ago, when the well was completed, an open concrete pit 7 feet in diameter was constructed from the ground level to a point 3 feet above the static water level. A horizontal pump was installed with an ordinary suction lift in the well and pumps at the rate of 50 g.p.m. This water from the limestone formation is highly mineralized, having a total solids content of 1500, the highest of any well water in the province. In Milverton, there was no alternative source of supply. The installation is economical as it is situated near the centre of the village. A surface reservoir was constructed at the pump house for a reserve supply and for fire service.

In the *Town of Hespeler*, water has been obtained for many years from two rock wells constructed in the Guelph dolomite. Originally the artesian flow from the wells, which are 96 feet deep, and which discharge into the surface reservoir, was sufficient to supply the town. Five years ago a deep well pump was installed in one well casing, having a capacity of 250 g.p.m. The flow from the other well is small.

In order to have a duplicate source of supply and duplicate pumping equipment, another rock well has been recently put down to a depth of 100 feet, 10 inches in diameter. The upper 50 feet was drilled out 12 inches in diameter, and a 10-inch casing was installed with a cement seal at the bottom to shut off the possibility of obtaining water from the upper levels containing objectionable organic matter.

There has been a certain amount of sulphur in the Hespeler supply, and it is expected that aeration will overcome this complaint. The water from the wells, as a matter of safety, has been chlorinated for some years. The wells are located near the river. And chlorination has been advisable in rock wells of this type where the water comes from fissures or crevices in the rock formation.

The original well casings were 8 inches in diameter. The bowls of the deep well pump, installed five years ago in the first well, have nearly the same diameter as the casing, making it impossible to put in a small air line to determine the drawdown when the pump is operating. The minimum diameter of casing for a deep well pump of 250 g.p.m., should be at least 10 inches; and the new well has this diameter, in which the new deep well pump can be readily installed.

In the *Town of Seaforth*, there are three rock wells in the limestone

formation having depths of 160, 202 and 244 feet. In 1922 the two shallower wells were equipped with deep well plunger pumps having capacities of 40 and 75 g.p.m. The larger and deepest well has more recently been equipped with a deep well turbine pump having a capacity of 200 g.p.m. The static water level is 35 feet below the surface. When pumped at the rate of 200 g.p.m., the drawdown is 82 feet.

In the *Town of Penetanguishene*, there is a good example of a municipal supply obtained from a shallow gravel formation with what might be called "home-made" wells. There are four wells consisting of 16-inch casings, 52 feet deep. The lower 20 feet of the casing in each well is perforated with ½-inch diameter holes, and the casings are open at the bottom. The depth of the gravel stratum is 12 feet and is below an impervious stratum of hardpan. The wells have an artesian flow into a surface reservoir. The horizontal centrifugal pumps are connected up in such a way that a suction lift can be put on the wells. In the installation and development of these wells, very fine sand was encountered and removed by air lift. When work was completed, these wells had virtually a natural gravel wall or envelope around the screens. The wells produce from 475,000 to 700,000 g.p.d., depending upon the consumption. It is an economical installation and has given good service.

Newmarket is a town that has never had an abundant supply of water for municipal purposes. Deep wells, located near the pump house, have been the source of supply. Many well casings have been put down, some to a depth of 290 feet, two of which only at the present time are productive. A tight fine sand formation with fine gravel and a certain amount of silt and clay supplies water grudgingly. It has been frequently necessary to clean out the fine sand and gravel that has heaved in the casings for the purpose of restoring their capacity. At the present time most of the supply comes from one 6-inch well finished in fine gravel at a depth of 286 feet. This well has an artesian flow into a surface reservoir.

Last year a contract was awarded for a gravel wall well for a guaranteed supply of 300 g.p.m., with a provision in the contract that if 200 g.p.m., were obtained, the Contractor would be paid on a pro-rata basis. The contract price is $8,400.00 for well and pumping equipment. The Contractor guaranteed the quality of the water bacteriologically. The chemical quality of the water is guaranteed to the extent of being at least equal to the chemical quality of the present

supply. In this way the municipality assumed some of the hazard in development, both as to quantity and quality, resulting no doubt in a lower contract price than would otherwise have been the case. A new gravel wall well has been put down to a depth of 157 feet in a location 1800 feet northeast of the existing wells. The well consists of a 24-inch outer casing installed to the top of the first water-bearing stratum at a depth of 105 feet. A 12-inch inner casing and two 12-inch shutter screen sections with $\frac{1}{8}$-inch openings, each 15 feet long, have been installed. The screens are located in the water-bearing strata between the depths of 105 to 120 feet, and 140 to 155 feet. In the installation of this gravel wall well, the rotary clay-seal method was used. It would appear that this method is a positive type of construction to obtain a definite placement of a continuous artificial gravel wall around the outside of, and for the full depth of the screens. In the development work of surging and pumping, and the washing out of the mud wall that was put in before the screens were lowered into position, 68 cubic yards of pea gravel, most of it well-rounded, uniform pea gravel from Cape May, were put in the lower formations around the screen sections. From calculations that we have made, this would mean that the artificial gravel wall has a diameter of about 7 feet, or is more than 3 feet thick around the screen. This should ensure an ample percolating and infiltration continuous gravel wall around the screens to lower the velocity of the water as it approaches the screens, and serve as an efficient filter to shut out the fine sand. This method is, we believe, an effective way to separate the water from the fine sand formation. There may be difficulty in washing out all the mud wall put in by the rotary clay-seal method before the casings were lowered into position, but from the fact that 68 cubic yards of pea gravel were put in around the screens, it would indicate that the mud wall has either been completely washed out, or disturbed in such a manner as not to interfere seriously with the flow of water into the well. The well is not yet completed, but has been tested to a capacity of 200,000 g.p.d. The Contractor intends to go deeper to increase the supply.

Newmarket is an illustration of a town that must develop and obtain water from the lower water-bearing horizons, or face the problem of obtaining a much more costly supply elsewhere. And it would seem that when the well, now under construction, is completed, an alternative source of supply will have been obtained at a reasonable cost. It is difficult during the process of installing a well

to obtain a representative sample of sand in the formations encountered, but, from analyses we have made, the sand stratum has an effective size of 0.41 m.m. and a uniformity coefficient of 2.86. The consumption of water in the town, by virtue of a fully metered system, and as a result of careful periodic surveys to discover water leakage, is only about 33 gallons per capita per day, which is very low for a municipality of 3500 with a complete sewerage system. The normal daily consumption is 110,000 gallons.

In the construction of a gravel wall well at Bradford, large boulders interfered with the work. In the first casing that was put down, such a nest of boulders was encountered that it was necessary to pull out the casing, move some distance laterally, and start over again. This well consists of a 16-inch outer casing to a depth of 82 feet, and a 10-inch inner casing to a depth of 112 feet, of which the lower 20 feet consists of a shutter screen. The well was drilled out originally to a depth of 160 feet, but the lower 48 feet was filled in with gravel because no further water-bearing formations were encountered below a depth of 112 feet. A thick artificial gravel wall surrounds the 20 foot screen. The stratum consists of fine sand with streaks of clay.

In the surging and development of this well, large quantities of fine sand and silt were pumped out and pea gravel was fed into the annular space of 3 inches between the two casings to take the place of the silt and sand so removed. The well has a small artesian flow, and when pumped at the rate of 160 g.p.m., the drawdown was 35 feet. The deep well pump is set at a depth of 69 feet. This well has ample capacity to supply the municipality, and has a pulley on the vertical shaft to which a traction engine can be belted if the motor is shut down for repairs.

The *Town of Barrie* obtains an abundant water supply from five artesian wells, three of which are 8 inches in diameter, and two 10 inches in diameter, all located within a radius of 350 feet from the pump house. The depth of the water-bearing stratum varies from 125 feet to 130 feet. A simple type of well collects the water from the formation consisting only of the casings resting on the gravel bed with open ends. There are no screens or perforations in the casings.

When the casings were driven to the required depth, the water flowed abundantly at the surface and brought up large quantities of fine gravel and sand. When the water became clear, it was piped to the surface reservoir in which the wells discharge by artesian flow. The bed of gravel at the end of the casing forms the screen, and has not clogged up for a period of years to appreciably lessen the supply.

The water in the reservoir is aerated by passing over diffuser plates inserted in the bottom of the reservoir. A vacuum priming pump in the pump house is also used as an air compressor by opening and closing certain gate valves in the piping arrangement. The aeration of the water has reduced the iron content of Fe_2O_3 from 0.30, to 0.24 p.p.m. By aerating the water, it has increased the amount of dissolved oxygen, and has removed certain complaints that existed about bad-tasting water in certain parts of the town, especially at the dead ends. Vegetable growths in this water apparently caused trouble on account of the lack of oxygen and presence of iron, and aeration has greatly improved the quality of the water. By changing FeO to Fe_2O_3 by aeration in the reservoir, red-water troubles have practically disappeared from tap water. Usually iron is not troublesome in water when less than 0.4 p.p.m. When in greater quantity than 0.5 p.p.m. not only is aeration necessary, but usually also filtration to remove the iron oxide.

In the *Village of Ancaster*, a supply of water was obtained from a gravel wall well installed near the centre of the village. The well consists of an outer casing 12 inches in diameter driven down to the top of a formation of gravel, and sand, at a depth of 50 feet. An inner 8-inch casing was driven down and bailed into the top of the rock formation at a depth of 60 feet. The screen is 10 feet long at the end of the 8-inch inner casing. In developing the well, a plunger or swabber was used to draw out the fine sand and silt in the formation, during which operation pea gravel was fed down in the annular space between the casings. The static water level in the well is 38 feet below the surface. When pumping at the rate of 100 g.p.m., the drawdown was only 5 feet, giving a specific yield of 20 g.p.m., per foot of drawdown, which is high for a well of this size. In setting the pump at a depth of 55 feet (in the screen section) the inner casing was removed from the well above the screen.

In *Almonte*, the town obtains the municipal supply from two rock wells located within 100 feet of each other. The wells are 111 feet deep and are 10- and 12-inch in diameter. Water is obtained from a natural underground reservoir in the fissures and crevices of the hard limestone formation. The static water level is 32 feet from the surface. The rock formation is within 5 feet of the ground level, but very little water was encountered in drilling until the depth of 100 to 110 feet was reached.

Vertical motors are connected with deep well turbine pumps set at a depth of 90 feet and discharge water directly into the distribution

system against standpipe pressure. The motors are automatically controlled for starting and stopping according to the level of the water in the standpipe, and require little attention by the operator. The capacity of the pumps is 125 and 150 g.p.m. respectively, and the drawdown of the larger well is 20 feet when being pumped at the rate of 150 g.p.m.

The supply of water is sufficient for the municipality and duplicate wells and pumping equipment is an additional factor of safety to ensure continuous supply.

On account of limited space, reference is made only to the salient features in connection with some other deep well installations in the province.

In the *Town of Ingersoll*, there are two 12-inch rock wells near each other in the pump house that were put down in the limestone to a total depth of 463 feet. The static water level is 37 feet from the ground, and the drawdown when pumping at the rate of 500 g.p.m., from one well is 34 feet, giving a specific capacity of 15 g.p.m. per foot of drawdown. The water contains hydrogen sulphide which is removed by aeration through nozzles at the top of the standpipe.

The *Town of Brampton* has a splendid gravel wall well having an outer casing 38 inches in diameter, and an inner casing 26 inches in diameter, put down to a total depth of 135 feet, and obtaining water from a gravel and sand stratum. The specific capacity of this well is 44 g.p.m. The deep well turbine pump is operated by a vertical motor, and is also connected for alternative power to a gasoline engine by means of a tex-rope drive connected to a vertical power take-off shaft from the gasoline engine.

The *City of Kitchener* with a population of 31,000 obtains all the municipal supply from wells of various types of construction in the sand and gravel formations, and has a supplementary supply from rock wells.

The *City of London*, where we are holding this Convention, has had a long and varied experience with ground water developments, and is also the largest city in Ontario obtaining the entire domestic supply from springs and wells. Approximately 5,000,000 g.p.d. are pumped to supply a population of 75,000 people. In addition to this, 500,000 g.p.d. of untreated water is pumped from the Thames River to certain industrial consumers, the largest of which is the Railway Company. The domestic consumers are completely metered. The ground water is obtained from springs and from deep wells at depths varying from

100 to 130 feet in the glacial drift deposits near the city. All the domestic supply is chlorinated. There is no economic alternative source of supply, and, in later years, the deep well system has been consistently developed to provide more water as required.

The low per capita consumption of 60 g.p.d., is a good illustration of how the consumption in a large municipality can be kept down to a comparatively low figure by having all the services metered, by careful periodic surveys to locate any leakage in the distribution system, and by educating the people over a period of years not to waste water.

In the *City of Stratford*, all the water is pumped from rock wells by means of an air lift system. The water is hard.

The *Township of Etobicoke* has two gravel wall wells that supply water for the municipality. The water is high in iron containing 1.4 p.p.m., and has a total hardness of 320 p.p.m. It is the only municipal waterworks plant in Ontario that has a combined zeolite water-softening and iron-removal plant.

Many of the deep well supplies in the province, especially supplies from shale, limestone and dolomite, produce very hard waters ranging from 250 to well over 400 p.p.m. There is a field for improvement in these hard water municipal supplies by installing water-softening plants. Soft water supplied to the consumers results in a substantial saving in the consumption of soap, and is much more acceptable and satisfactory for household purposes. Where the exacting requirements of certain industries for soft water are concerned, it is essential that soft water be supplied or the industries must locate elsewhere or install their own softening plant.

GENERAL

In new ground water developments or installations to supplement the existing supplies, each well is a separate and distinct problem and should be carefully studied on its merits. In the past fifteen years, there has been a great advance in ground water hydrology. The scientifically constructed gravel wall well, originated primarily to separate water from fine sand formations, has contributed much to increase confidence in ground water supplies, and has produced water in certain areas in much larger quantities than was hitherto thought possible. In the smaller municipalities, where the quantity of water required is not large, it would seem that in good gravel formations it is not necessary to go to the expense of installing this type of well.

In certain localities, where there are obvious surface water supplies, ground water developments should not be seriously considered. I recall the case of a village where considerable money was spent in sinking a deep well to a depth of 500 feet to secure water for a municipal supply, when there was a spring of beautiful clear water of ample capacity within the boundaries of the village which was finally used for the municipal supply.

Ground water supplies are permanent and reliable, if the wells are not over-pumped, assuming that the wells have been properly constructed. In operating a new supply system, careful observations should be made of the drawdown of the well and the ground water levels in the area affected. If the well or wells are over-pumped, in time the natural underground reservoir becomes depleted, and finally the supply from the wells becomes a failure, for the intake or recharge is not equal to the withdrawal or pumpage. A careful quantitative investigation would establish the perennial safe yield of a system of wells.

In the contracts for a guaranteed supply the final tests should indicate clearly that the rate of pumping is not greater than the recharge, and a considerable period of time might be necessary to establish these facts conclusively.

Ground water development is an intricate and difficult undertaking on account of the variety and complexity of geologic formations, and of the irregular manner in which the glacial drift has been deposited in various sections of the country.

In the work of installing deep wells of various types, we are all dependent to a large extent on the type of equipment available, experience, patience, skill and judgment of the well drillers. The elements of risk and hazard are important factors in work of this character, even after a careful survey has been made of the geologic formations, the typography and water table, the replenishment area, sanitary problems and convenience and economy of location of wells.

In a "No Water-No Pay" guaranteed supply contract, there should be a definite understanding as to the distance a permanent well may be put down from the distribution system, and test holes should be put down for exploratory work at locations mutually agreed upon. It would seem to be proper for a municipality to pay at least a part of the cost of such exploratory work. If a large test hole is advisable in the preliminary work, where a pumping test can be made, chiefly to obtain a representative sample of water for chemical analysis, the

municipality should pay a price per lineal foot to cover the cost of this test hole, as set forth in the original tender.

Regarding the chemical quality of the water, it should be satisfactory to the municipality and to the Department of Health of the Province of Ontario as a result of preliminary tests, but when once the permanent well or wells are proceeded with then the municipality could very well assume the hazard of the chemical quality. From experience, it would seem that, when the municipality assumes some of the hazard in the installation of a supply system from deep wells, this fact is usually reflected in a lower contract price for a guaranteed supply than otherwise would be the case.

The clauses for payment in a guaranteed water supply contract have perhaps been too favorable to the Contractor. One might even assume that in a "No Water—No Pay" contract, the municipality might expect the Contractor to locate, develop and produce water of a satisfactory quality and quantity before asking for any payment. To the writer's knowledge this has not been done in any municipality in the province. But, if this stipulation is not a part of the contract, payments should be made on the certificate of the Engineer on the basis of work done and materials supplied, the same as for ordinary ground level work. No payment should be made until a satisfactory surety bond has been furnished which guarantees the supply of water in accordance with the contract or the refund of the money. In Almonte the drilling company went into bankruptcy before the contract was completed, and the Surety Company completed the contract. If money is to be refunded, it is usually a matter for decision by legal opinion.

The guarantee bond should guarantee the quality of the water bacteriologically for at least one year, and also the minimum guaranteed quantity of water, having regard to the rate of pumping and drawdown, and also the intake or recharge. In a gravel wall well contract, it should provide that the work will be satisfactorily performed, and, that if the well is a continuous "sander," it would not be acceptable until clear water free from sand and silt is produced.

The development of the turbine deep well pumps in recent years to a high degree of perfection has made deep well pumping equipment comparable in efficiency and reliability to horizontal centrifugal pumps.

The type and efficiency of pumping equipment are important factors in ground water supplies. The diameter of casings is closely

associated with this problem. For deep well turbine pumps and motors using 25 cycle current, a 6-inch pump of approximate capacity of 100 g.p.m., has a wire to water efficiency of approximately 54 percent; an 8-inch pump of 200 g.p.m., 65 percent; a 10-inch pump of 500 g.p.m., 72 percent; a 12-inch pump of 850 g.p.m., 72 percent; and an 18-inch pump of 2100 g.p.m., 75 percent. Therefore, the larger the casing, the more efficient is the pumping equipment which can be installed.

No well casing for a municipal supply should be less than 8 inches in diameter, and 10- or 12-inch would be better.

For the smaller installations requiring from 20 to 75 g.p.m., usually deep well, reciprocating, double-acting pumps are to be preferred, especially if the pump has a deep setting in the well. In a casing $4\frac{3}{4}$ inches in diameter which recently came under our observation, the maximum quantity of water that could be pumped by a plunger pump of this type was about 20 g.p.m.

The air lift pump, though inefficient, has its place in various types of installations. It is especially useful in crooked wells or in small wells widely distributed where it is economical to obtain the air from a central compressor house. Where aeration is advisable, the air lift system for pumping may supply sufficient air for this purpose. For the proper operation of air lift pumps, wells of considerable depth, on account of the submergence required, are necessary.

In conclusion, I wish to make a few remarks about water supply systems as a whole. If sufficient water is obtained from wells of satisfactory quality, a storage reservoir is usually required at the surface of the ground and a motor-driven and gasoline driven centrifugal pumps of large capacity for fire-fighting purposes. The deep well supply system, therefore, is only a part of the complete supply works. In comparing deep-well supplies with surface-water supplies, these supplementary works should be kept in mind in considering the merits or advantages of one source of supply with another. Long pipe lines to well supplies may be a large item in the ultimate total cost.

There are many engineering problems to be considered in these complete supply systems, each of which should be studied as an individual problem on its merits before arriving at a conclusion. In all proposed ground water supplies and supply systems, there is a field of useful service to municipalities for waterworks engineers.

(*Presented before the Canadian Section meeting, March 27–29, 1935.*)

FILTER WASHING EXPERIMENTS AT THE CHICAGO EXPERIMENTAL FILTRATION PLANT

By Herbert E. Hudson, Jr.

(Junior Sanitary Engineer, Department of Public Works, Chicago, Ill.)

Although filter bed troubles have been widespread for a long time, no effective method of avoiding them has been widely adopted. It was thought the high velocity wash would prove a panacea which would forever consign clogged beds, surface cracks, mud-balls, and sand shrinkage into limbo, but extended experiments at the Chicago Experimental Filtration Plant have disproved this. The tests have proved that the surface-wash as a means of keeping filter beds perfectly clean is entirely effective.

There is still some controversy whether filters should be kept clean or not. Many filtration engineers cling to the idea that coating on the sand grains is necessary to adsorb the impurities from the water. On the other hand, Hulbert and Herring (1) have presented very convincing data to show that clean filter beds can effect good bacterial removal, and that it is altogether desirable to keep filter sand clean. The Chicago experiments, too, indicate that clean filters *can* produce a satisfactory effluent. In fact, in some cases clean filters produce better water than filters containing dirty sand, and clean filters can be operated in such a way as to produce a satisfactory effluent at all times.

By satisfactory effluent, Chicago means a filtered water containing no flocculated turbidity and less than 0.2 p.p.m. of total turbidity. It is realized that this is a difficult requirement for all cities to meet, but experience has shown that it is not too rigid a standard for a filter plant treating Lake Michigan water at Chicago.

This paper makes no detailed attempt to show the detrimental effect of accumulated dirt in the sand upon the condition of the filter bed itself. Some operators question whether it is worth while to go to additional trouble and expense in order to keep filter beds perfectly clean. Perhaps there are plants where perfectly clean sand is not essential, but most of those who have encountered filter bed troubles know that in a dirty filter the possibilities of sand and gravel mixing

and underdrain clogging are ever present. With a clean, properly designed sand-bed, these are not possible. All this has been discussed by Baylis (2, 3, 4, 5). Surely any condition which threatens to prevent the continued operation of a filter is not desirable. Figure 1 shows the condition of a filter which had become dirty through no neglect of the plant operator. How can such a bed be expected to continue producing good water?

Not all filter bed troubles can be blamed on the method of washing. It should be emphasized that water should always be properly prepared for filtration. This includes use of the optimum chemical dosages, proper flocculation, and proper sedimentation to prevent

FIG. 1. VIEW OF RAPID-SAND FILTER BED SHOWING CRACKS AND MOUNDING CAUSED BY ACCUMULATED COAGULATED MATTER

overloading the filters. For example, one could hardly expect the filter sand to remain clean in a plant where the water does not coagulate completely until it reaches the sand. Good washing may tend to overbalance deficiencies in treatment, but even a perfectly clean filter can fail to clarify a poorly prepared water.

The equipment at Chicago has been described carefully in other papers (6, 7, 8), so no detailed description will be given in this article. In the experiments, the following were used: a concrete filter, area—100 square feet; steel tank filters, area—10 square feet apiece; and glass-tube filters, area—0.17 square foot apiece. The most extensive experiments were run on filters containing 2 feet of Muscatine sand

having an effective size of about 0.48 mm. This sand was used in all the rapid-sand filters excepting the glass-tubes.

During all the experiments herein described, the water was prechlorinated with a dosage sufficient to produce a chlorine residual averaging about 0.5 p.p.m. Experience showed that time spent in making bacterial tests of the filter effluents for analysis of experiments was time wasted. Dead bacteria generally give negative tests. Bacterial analyses of more than 2,000 samples of water which had been prechlorinated and filtered through filter beds in poor condition showed no B. coli in any sample. Of far greater value was the quickly made turbidity determination since it showed the efficacy of coagulation and gave warning of filter bed troubles.

TABLE 1

Methods of washing used on plant filters (washing series)

FILTER NUMBER	METHOD OF WASHING	TIME OF WASHING	RATE OF WASHING IN G.P.M. PER SQUARE FOOT	
			Backwash	Surface
		minutes		
3	Back only	4	12	
4	Back only	3	15	
5	Back and surface	3	12	2.5
6	Back and surface	3	11	2.5
7	Back only	2	23	

Five of the steel-tank filters were put into service in November, 1929, to find the effect of the method of washing on filtration. These filters were operated at a rate of filtration of 2 g.p.m. per square foot. Filters 5 and 6 were equipped with surface-wash piping. There were six $\frac{3}{16}$-inch jets to each filter, located 2 inches above the unexpanded sand, and pointed downward at an angle of 30 degrees from the horizontal. All filters received the same water. The only difference between them was the manner in which they were washed. Table 1 gives data on the methods of washing used. These filters were operated in this manner until October, 1931, and again in December, 1931.

CONDITION OF THE FILTER BEDS

The rates of washing were checked periodically and inspections of the sand beds were made at the same time. Notes were kept of the

observations made. The data from these are summarized in table 2 which has been prepared because a detailed description of the condition of some of these filters would take up too much space.

Chlorinated Lake Michigan water was used for washing the filters, and it contains some particles large enough to clog the $\frac{3}{16}$-inch holes used in the surface-wash system. Table 2 shows that there was some pulling away from the side-walls in Filter 5. There was difficulty with the surface-wash jets clogging in this filter. The surface-wash holes on Filter 6 clogged to a lesser degree, although during June and July, 1931, they were badly clogged. The effect of this clogging is reflected in the condition of the filter on July 27, 1931.

TABLE 2

Condition of rapid-sand filters washed in various ways

DATE	FILTER NUMBER				
	3	4	5	6	7
November 21, 1929	S	MC	M		M
March 3, 1930	MSP	MP			MP
May 21, 1930	M	MS			M
October 1930	MP	MP	P	P	MP
May 21, 1931	MSPC	MP	P		MP
July 27, 1931	MSP	MSP	P	MSPC	MP
October 5, 1931	MP	MP	P	C	MP
November 2, 1931	MS	MS	P		MP
January 5, 1932	MSP	MSP	P	P	M

"M" indicates mud-balls; "S" indicates surface cracks; "P" indicates pulling away from the sidewalls; and "C" indicates clogged areas.

Where no letters appear under a filter for any date of inspection, that filter was in perfect condition.

It is difficult to summarize information such as is contained in table 2, for such a summary cannot easily tell the degree to which a certain condition occurred. For instance, from the summary one could not tell accurately whether Filter 4 was equal in condition to Filter 7. If these filters were to be arranged in the order of their condition, those who observed them would arrange them thus, from best to poorest: 6, 5, 7, 4, 3. Figures 2, 3, 4, 5, and 6, show the condition of the filters on May 27, 1930. From these illustrations a good idea of the condition of the filters can be obtained. These illustrations will serve to show the meaning of table 2.

From these observations it is clear that the surface-wash system

kept the sand in these filters cleaner than any rate of backwashing alone did. It is also apparent that a rate of wash of 23 g.p.m. per

FIG. 2. FILTER NO. 3, MAY 21, 1930
A low rate of back-wash was used

FIG. 3. FILTER NO. 4, MAY 21, 1930
15 g.p.m. per square foot back-wash was used

square foot kept Filter 7 in better condition than Filters 3 and 4 which were washed at 12 and 15 g.p.m. per square foot, respectively.

Further experiments were run on a number of different filtering materials in glass-tube filters. The materials tested were: sand from

FIG. 4. FILTER NO. 5, MAY 21, 1930
The surface-wash was used with the back-wash

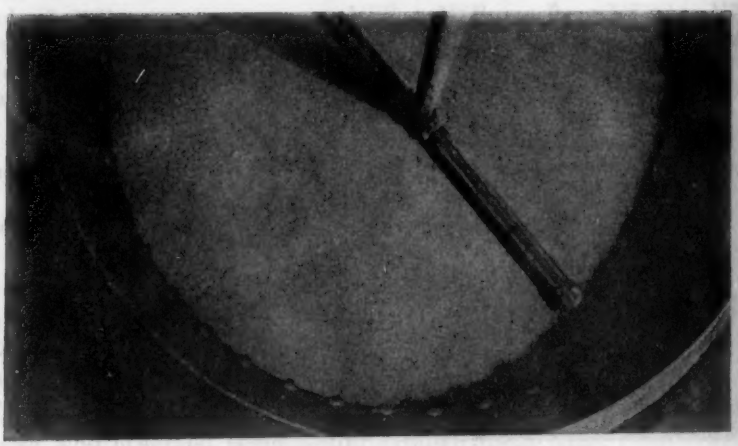


FIG. 5. FILTER NO. 6, MAY 21, 1930
The surface-wash was used with the back-wash

Muscatine, Iowa; crushed quartz from Copley, Ohio; Minchar; Magnetite; anthracite coal; iron ore; and crushed glass. These were

used to determine the effect of porosity and shape of filtering material upon filtration.

The materials in the glass-tube filters were washed at such a rate that 50 percent expansion of the bed was produced. This was done both when the surface washes were used and when they were not. Back wash was used at all times, but at a lower rate when the surface-wash was used.

After operating the filters for some time, it was found that mud-balls had formed in each one, and it was feared that this might affect the filters runs. Consequently, these filters were equipped with one

FIG. 6. FILTER NO. 7, MAY 21, 1930
High velocity back-wash used, but surface-wash not used

small surface jet each. The mud-balls deep in the beds were broken up mechanically before starting use of the surface-washes. From July 1 to July 23, 1934, the surface-washes were used during each wash. No mud-balls formed, no coating occurred and all these materials remained in perfect condition.

On July 23, the surface-washes were taken out of service. Mud-balls began to form, and they were present in each filter by November 13. Some filters were badly clogged at the gravel, and some pulled away from the sidewalls. On November 13, they were cleaned up, and the use of surface-washes resumed. The filters stayed in perfect condition until December 23, 1934, when the surface-washes were again discontinued. Ten days later, there were mud-balls in each

filter, and in one they had settled onto the gravel. Pulling away from the side-walls also occurred.

Lake Michigan water does not cause filter bed troubles to so great an extent as some waters. Hence, none of the filters got into such bad condition as to require overhauling.

The idea exists among some water works engineers that prechlorination will assist in keeping filter beds clean. During all the washing tests, the water was highly prechlorinated. In the absence of comparable data, it is thought that the effect of prechlorination is a minor one. Certainly enough trouble was experienced in operating filters according to standard practice, with prechlorination, to prove that it will not cure filter bed troubles.

TABLE 3

Effect of method of washing on lengths of filter runs

	FILTER				
	3	4	5	6	7
Hours filter run.....	42.2	41.8	32.5	33.1	42.8

EFFECT OF METHOD OF WASHING ON LENGTHS OF FILTER RUNS

During the period that data were collected on the conditions of the 5 steel-tank filters which were washed in different ways, loss of head readings were taken regularly, and from these the lengths of the filter runs have been computed. Table 3 gives the weighted averages of the filter run lengths. The length of run was defined as the time in hours required to reach 8 feet loss of head.

The filters equipped with surface-washes gave shorter runs than those which were not. Probably this was because cracking of the bed surface and pulling away from the side-walls did not occur to any material extent in the surface-washed filters, hence there was no opening up of additional filtering area or lessening of the effective depth of bed as the filter runs progressed.

The data on lengths of filter runs show that clean filters give shorter runs than those containing dirty sand. Chicago water has little tendency to coat sand grains, although the coagulated matter sticks to sand grains and causes mud-ball formation. However, at Detroit and at Baltimore, it was found that the initial loss of head was greater for dirty, coated sand than for clean. This might be taken as an

indication that the filter runs would be longer on clean sand there. When coated sand in a filter compacts under filtration heads the voids are thought to become smaller, thus increasing the resistance of the material to filtration. Hence, filter runs may, under certain circumstances, be lengthened by cleaning the sand. Only careful experiments on a water which forms coatings of the sand grains will tell.

EFFECT OF METHOD OF WASHING ON CLARITY OF FILTER EFFLUENTS

Daily samples were taken from the five filters of the washing series. They were analyzed for turbidity on the Baylis turbidimeter, which permits readings to 0.1 p.p.m. within the range 0.0 to 2.0 p.p.m. Table 4 shows the monthly averages of these measurements. For convenience of analysis, the data have been divided into two periods: December, 1929 to February, 1930 inclusive; and March, 1930 to December, 1931 inclusive. Since the data for October and November, 1931 were incomplete they have been omitted.

The period December, 1929 to February, 1930 was a time of weak flocculation. It was impossible at that time to produce a coagulation which had strength enough to resist being pulled through the filters when the loss of head approached 8 feet. Clear water could have been produced by washing the filters at a loss of head less than 8 feet without producing too short filter runs. The tests given in Table 4 are averages of turbidity determinations on "grab" samples, which were taken at all losses of head up to 9.0 feet. They show higher turbidities than should be permitted in operation of a plant producing a public water supply. During this time of weak flocculation, many analyses were made with a floc detector. These data showed that the surface-washed filters gave longer runs before passing floc than those which were simply backwashed. Table 5 is a typical group of tests during this time. The observations made on filters washed in different manners during times of weak flocculation showed conclusively that the surface-washed filters produced the clearest water with the longest filter runs.

Turbidity tests made on the effluents of the washing series filters during the period March, 1930 to December, 1931 indicate that, for normal flocculation, there is no difference in the quality of the effluents from filters washed in various ways. The turbidity content of the filtered water averaged 0.02 p.p.m. in each case. These steel-tank filters were washed with chlorinated raw water which has

an average turbidity of about 10 p.p.m., and sometimes, some of this turbidity may have appeared in the filtered water. It is probable

TABLE 4

Turbidities of filter effluents, washing series

DATE	NUMBER OF SAMPLES PER FILTER	TURBIDITY IN VARIOUS FILTER EFFLUENTS, IN P.P.M.				
		3	4	5	6	7
1929						
December	17	0.46	0.40	0.12	0.18	0.16
1930						
January	26	3.05	1.83	0.50	0.30	2.13
February	22	0.38	0.33	0.05	0.06	0.07
March	26	0.02	0.01	0.00	0.02	0.00
April	26	0.01	0.01	0.02	0.00	0.00
May	23	0.02	0.02	0.05	0.02	0.03
June	25	0.05	0.06	0.03	0.03	0.04
July	25	0.00	0.04	0.01	0.00	0.01
August	26	0.01	0.02	0.02	0.01	0.01
September	25	0.04	0.06	0.06	0.05	0.05
October	26	0.01	0.01	0.00	0.00	0.00
November	22	0.02	0.00	0.03	0.04	0.03
1931						
January	25	0.02	0.02	0.00	0.03	0.00
February	23	0.00	0.00	0.00	0.00	0.00
March	26	0.06	0.03	0.06	0.03	0.06
April	25	0.04	0.03	0.02	0.02	0.06
May	24	0.00	0.00	0.00	0.00	0.00
June	23	0.01	0.02	0.00	0.01	0.00
July	25	0.00	0.00	0.00	0.00	0.00
August	21	0.00	0.00	0.00	0.04	0.00
September	24	0.01	0.01	0.02	0.01	0.00
December	25	0.04	0.03	0.06	0.03	0.04
Average, December, 1929 to February, 1930, inclusive		1.30	0.85	0.22	0.18	0.79
Average, February, 1930 to December, 1931, inclusive		0.02	0.02	0.02	0.02	0.02
Average, December, 1929 to December, 1931, inclusive		0.19	0.13	0.05	0.04	0.12

that, had these filters been washed with filtered water, the filtered turbidity would have averaged less.

The studies made indicate that the method of washing makes no difference in the clarity of the filtered water when there is normally strong coagulation, but that surface-washing has a distinct advantage when the floc is weak. When weak flocculation occurs, it is desirable to have the whole bed in perfect working order to insure a good

TABLE 5

Flocculated matter passing filter beds

Water treated with about 7 p.p.m. aluminum sulfate (27 minutes mixing and 3.1 hours settling).

DATE	HOURS IN SERVICE	TURBIDITY SETTLED WATER GOING TO FILTERS	FILTERS											
			2		3		4		5		6		7	
			Loss of head, feet	Volume of floc	Loss of head, feet	Volume of floc	Loss of head, feet	Volume of floc	Loss of head, feet	Volume of floc	Loss of head, feet	Volume of floc	Loss of head, feet	Volume of floc
1930														
January 13. . .	0	25.0	1.5	0	1.1	0	1.1	0	1.1	0	1.1	0	1.1	0
	7	25.0	1.8	1.0	2.3	0	1.8	3.0	1.8	0	1.9	0.5	1.3	2.0
January 14. . .	24	26.0	2.8	52.0	3.9	35.0	3.6	40.0	4.4	8.0	4.9	6.0	3.5	25.0
	31	26.0	3.4	96.0	4.7	22.0	4.4	40.0	5.7	15.0	6.5	11.0	4.1	30.0
January 15. . .	49	17.0	4.6	80.0	6.7	40.0	6.5	65.0	9.0	12.0			6.2	30.0
January 16. . .	73	22.0	6.7	70.0	9.6	48.0	9.2	52.0					9.4	40.0
	79	22.0					9.7	60.0						

Loss of head at which flocculated matter began to pass, feet.	1.7	2.5	1.6	3.0	1.9	1.2
Length of filter run before flocculated matter began to pass, hours.	6	8	5	12	7	5
Time required for loss of head to reach 8 feet, hours	81	61	62	44	40	66

effluent. Several possible reasons for the easier passage of floc through a dirty filter are:

1. Higher rates of filtration through unclogged areas may tend to pull floc through the bed;

2. Short-circuiting of flow through cracks or places where the sand has pulled away from the side-walls may take place.

3. Possible derangement of the gravel may make the sand layer too thin in spots to be effective, or it may cause mixing of the sand and gravel, thus increasing the effective size of the sand;

4. Coating of grains and entrapment of fine grains in mud-ball and clogged areas may increase the sand's effective size.

EFFECT OF BUMPING AND WASHING RAPID-SAND FILTERS ON THE CLARITY OF THE FILTERED WATER

Reverse-flow agitation of rapid-sand filter beds is a fairly common practise, yet there is seldom any discussion of it among water works engineers. As a result, practically no data exists on the results produced by it. Generally a filter is bumped by closing the filter effluent and turning on the filter wash water for a few seconds, then placing the filter back in service again. The object of this procedure is to break up the tough mat of coagulated matter which has formed on top of the sand during short filter runs. The disadvantage of

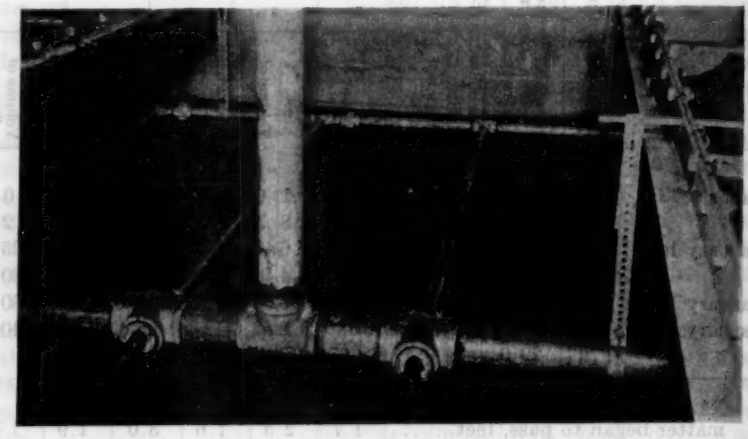


FIG. 7. FILTER NO. 2 AT THE CHICAGO EXPERIMENTAL FILTRATION PLANT
The surface-wash piping is immediately above the sand

bumping filters in this way is the possibility of damaging the rest of the bed at the same time the "schmutzdecke" is broken up. Also a turbid effluent may sometimes be produced.

In May and June, 1932, a number of special tests on the effect of bumping and washing one of the large concrete filters were made. This filter was free from mud-balls, clogging, surface cracks, and did not pull away from the side-walls. Surface-wash had been used at all times. Figure 7 is a view of this filter. Two methods of breaking up the surface mat on the filter were tried: first, reverse-flow agitation; and second, surface-wash agitation. Data were also collected on the effect of washing on the effluent of the filter. Turbidity samples were taken from the effluent of the filter at frequent intervals

after the filter had been subjected to each of the above procedures. The intervals were 0, 3, 6, 10, 15, 20, and 30 minutes.

When reverse-flow agitation was used, it was necessary to close the filter effluent during the agitation; when the filter was bumped with the surface-wash, the effluent was allowed to remain open, and the surface-wash valve was opened and closed, taking about 30 seconds for the whole process. This procedure was repeated each time the filter loss of head reached its maximum, until it was thought that the filter ought to be washed. It was found possible to increase the time between washes by more than 300 percent with an extremely small

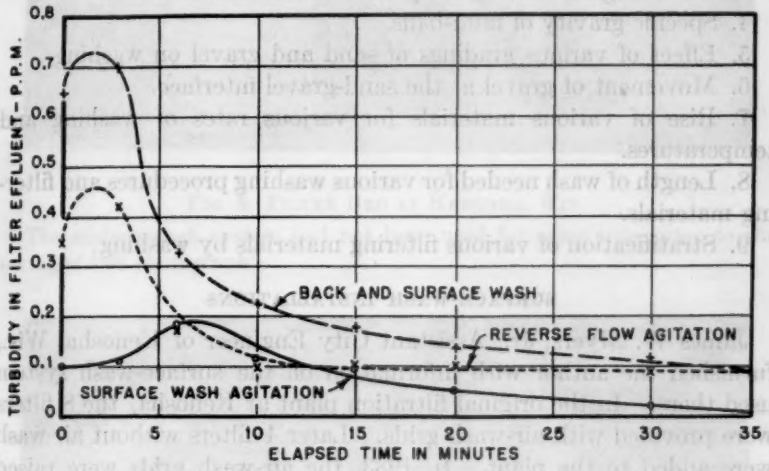


FIG. 8. EFFECT OF BUMPING AND WASHING FILTERS ON THE CLARITY OF THE FILTER EFFLUENT

increase in wash water use. The rewash was not used on this filter during these tests.

The results of the turbidity determinations on the effluent following these practises are shown in figure 8. The points plotted represent 23 sets of tests on surface-wash agitation, 6 on reverse flow agitation, and 6 on back-washing. Each test comprised 7 turbidity determinations. Ordinary back-washing produced an unsatisfactory effluent for about 12 minutes after washing, reverse-flow agitation for about 5 minutes, and the surface-wash agitation caused a maximum average turbidity of less than 0.2 p.p.m.

This is not intended to advocate the bumping of filters, but the data collected indicate that if the filters are bumped, surface-wash agitation

will provide a clearer effluent than reverse-flow agitation will provide. Further, with surface-wash agitation there is little danger of disturbing anything but the top few inches of the sand.

OTHER EXPERIMENTS

Numerous other studies have been made, are being made, or are contemplated, which are not discussed in this paper. Among these are:

1. Effect of shape of material on washing.
2. Effect of specific gravity of material on washing.
3. Use of high specific gravity materials in the filter bed.
4. Specific gravity of mud-balls.
5. Effect of various gradings of sand and gravel on washing.
6. Movement of gravel at the sand-gravel interface.
7. Rise of various materials for various rates of washing and temperatures.
8. Length of wash needed for various washing procedures and filtering materials.
9. Stratification of various filtering materials by washing.

SURFACE-WASH INSTALLATIONS

James W. Myers, Jr., Assistant City Engineer of Kenosha, Wis., furnished the author with information on the surface-wash system used there. In the original filtration plant at Kenosha, the 8 filters were provided with air-wash grids. Later 4 filters without air-wash were added to the plant. In 1933, the air-wash grids were raised above the sand, connected with a water pressure line, and put into service for surface-washing the filters. The surface-wash system has proven such a success there that the 4 added filters are being equipped with surface-washes. At no time since beginning the use of the surface-washes has there been mud-ball formation in the surface-washed filters, while those simply back-washed have not stayed in good condition.

A test of the value of the surface wash was made at Kenosha, and Myers states,

> Both filters are equipped with surface-wash equipment and both filters were clean at the start of this test. The two photographs tell the complete story (figures 9 and 10). At the end of this period, which was nineteen days, Filter 3 carried a considerable deposit of surface dirt which could not be washed out by ordinary back-washing. An examination of the bed revealed the formation

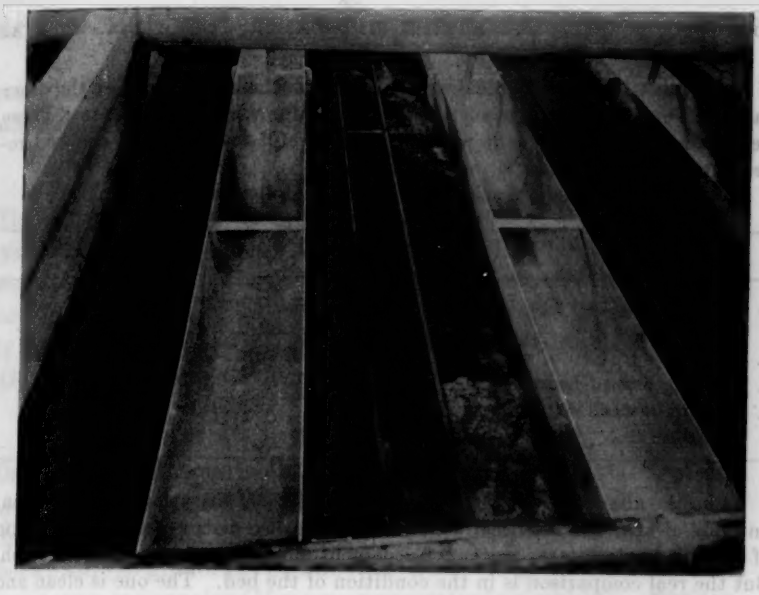


FIG. 9. FILTER BED AT KENOSHA, WIS.

The surface-wash system had not been used for some time prior to the taking of this photograph.

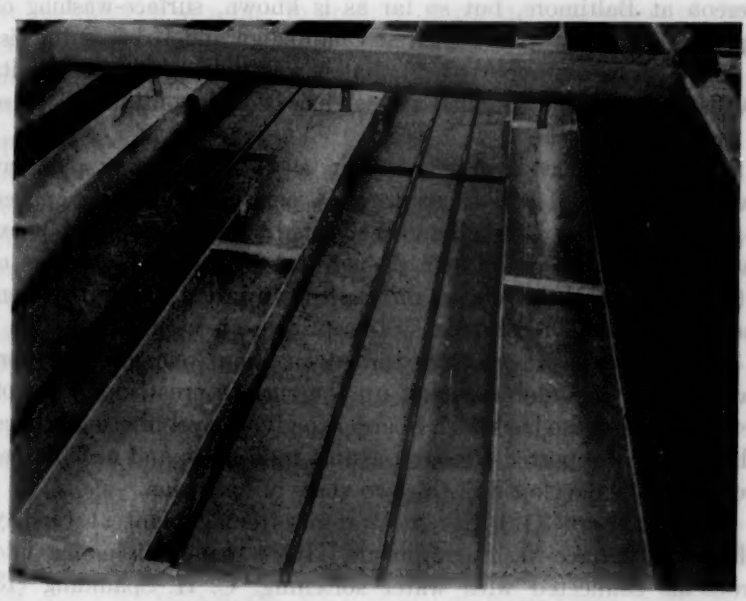


FIG. 10. FILTER BED AT KENOSHA, WIS.

The surface-wash system had been used on this filter

of mud-balls to a considerable depth below the surface. Filter 4, on the other hand, was entirely devoid of surface dirt except for a few little cinders. These were removed in subsequent washings. A careful inspection of the bed revealed a decided absence of mud-balls.

For your further information we submit the following data:

	FILTER 3	FILTER 4
1. Rate of flow through filters in gallons per day	1,055,232	1,088,000
2. Average duration of run	11 hrs., 32 min.	11 hrs., 30 min.
3. Average loss of head, feet	7.7	7.7
4. Percent of wash water to filtered water	4.4	5.1

From the above data you discover that the rate of filtration, length of run, and loss of head are practically the same. We also discovered that the ratio of wash water to filtered water is 0.7 percent more by using the surface wash. But the real comparison is in the condition of the bed. The one is clean and the other dirty.

Pearse and DeBerard experimented with the surface wash at Oakland, Calif. in 1907 and Baylis also conducted experiments thereon at Baltimore, but so far as is known, surface-washing of rapid-sand filters was first tried at Champaign and Urbana, Illinois, in 1915. Amsbary reported (9) that it was successful, but Babbitt (10) later stated that the surface-wash had been discontinued because it was uneconomical.

Surface-wash has been installed in a water softening plant at Evansville, Wis. (11). About 3 g.p.m. per square foot are applied to the sand surface through a grid situated 2 inches above the unexpanded sand. There are four $\frac{5}{32}$-inch jets for each 3 square feet of filtering area, and the jets are directed downward at 30 degrees from the horizontal. Distribution system pressure is used.

The surface-wash system is turned on about one minute before backwashing in order to break up the surface crust, and shut off before stopping the backwash in order to give a level filtering surface. The system is reported effective against mud-balls, and no filter bed troubles have developed in the two years of operation.

Goudey reports (12) that the water softening plant at Findlay, Ohio has surface-wash equipment. In a paper discussing filter problems connected with water softening, C. H. Spaulding (13) mentioned that surface-wash at a plant employing excess lime treat-

ment with recarbonation has proved valuable in keeping the sand clean and free from mud-balls.

ACKNOWLEDGMENT

The above work was done under the direction of John R. Baylis, Physical Chemist, to whom the writer is indebted for many suggestions and for some of the illustrations. The experimental work is done by the Bureau of Engineering, Department of Public Works, City of Chicago, Loran D. Gayton, Acting City Engineer and Arthur E. Gorman, Engineer of Water Purification.

SUMMARY

Descriptions of experiments on filter washing conducted at the Chicago Experimental Filtration Plant are given.

Backwashing filters failed to keep them clean, although high rates of backwash (23 g.p.m. per square foot) achieved greater success than customary or lower rates (15 g.p.m. per square foot or less). Surface-washing, in addition to backwashing, kept filters in very good condition when the total rate of wash was about 15 g.p.m. per square foot.

Glass-tube filters containing different materials formed mud-balls, when backwashed at a rate to produce 50 per cent expansion of the filtering material, during three separate periods. After each period, resumption of surface-washing kept the material in perfect condition.

Filter runs were shorter for the surface-washed filters than for those simply backwashed.

Clean filters passed flocculated turbidity less readily than dirty ones when there was weak coagulation. The filtered effluent turbidity was unaffected by the method of washing when there was normal or strong coagulation.

The available information indicates that it is desirable to keep filter sand as clean as possible.

Bumping filters produced less turbid effluents than did backwashing. Surface-wash agitation caused less turbidity in the filter effluent than reverse-flow agitation.

Information on surface-wash installations at several cities is given.

REFERENCES

(1) HULBERT, R., AND HERRING, F. W.: Studies on the Washing of Rapid Filters. Jour. A. W. W. A., 21: 11, November, 1929, p. 1445.

(2) BAYLIS, J. R.: Sand-Bed Studies at Montebello Water Filters, Baltimore. Engineering News-Record, 92: 13, March 27, 1924, p. 516.

(3) BAYLIS, J. R.: The Rapid Sand Filter Bed. Water Works and Sewerage, 77, 1, January 1930, p. 7.
(4) BAYLIS, J. R.: Keeping Rapid Sand Filter Beds in Good Condition. Water Works and Sewerage, 79: 4, April 1932, p. 124.
(5) BAYLIS, J. R.: A Study of Filtering Materials for Rapid Sand Filters. Water Works and Sewerage, 82: 1, January 1935, p. 20.
BAYLIS, J. R.: A Study of Filtering Materials for Rapid Sand Filters. Water Works and Sewerage, 82: 3, March 1935, p. 89.
(6) GAYTON, L. D.: Experimental Water Filtration Plant for Chicago Engineering News-Record, 100: 22, May 31, 1928, p. 861.
(7) BAYLIS, J. R.: The Experimental Filtration Plant of Chicago. Municipal News and Water Works, 76: 3, March, 1929, p. 89.
(8) BAYLIS, J. R.: Filter Sand Studies. Water Works and Sewerage, 79: 2, February 1932, p. 43.
(9) ARMSBARY, F. C.: Further Development of Iron-Removal Plant. Jour. A. W. W. A., 3: 2, June, 1916, p. 400.
(10) BABBITT, H. E.: Chlorine Cheaper Than Top Jets for Iron-Removal Plant. Engineering News-Record, 92: 14, April 3, 1924, p. 563.
(11) Design of New Water-Softening Plant. Engineering News-Record, 112: 19, May 10, 1934, p. 602.
(12) GOUDEY, R. F.: Water Purification Outside of California. Jour. A. W. W. A., 24: 6, June 1932, p. 822.
(13) SPAULDING, C. H.: Filter Problems and Water Softening. Jour. A. W. W. A., 24: 9, September 1932, p. 1352.

DISCUSSION

CHARLES H. SPAULDING (*Springfield, Ill.*): A good many years ago, the writer was given his first introduction to a gravity rapid-sand filter plant on the Panama Canal Zone. This plant was turned over to him with the beds badly fouled and showing all the symptoms which poor sand washing produces. One of the filters had had the sand and gravel entirely removed, and it was my duty to replace these and continue the removal and washing of sand from the other units. It was most disappointing to find, after going through this messy experience, that within a few weeks the filters were in about the same condition as when the sand removal had been started.

It is surprising how many years it has taken for filter operators to tackle this problem in the right manner. The most important contribution to the solution of the problem, in my mind, has been the use of experimental, glass tube filters in which the behavior of the filter bed in the backwashing is plainly visible. These glass filters have revealed the breaking-up of the surface scum in large fragments that settle beneath the surface of the sand and eventually reach the gravel layer. In many cases, no amount of velocity is

sufficient to carry away the flocculated material which has been compacted on the surface of the bed, unless it is first disintegrated. That part of the floc which has penetrated the voids of the sand in the regular course of filtration is easily moved in either direction; but that part which becomes agglomerated at the filter surface in a sticky mass needs first to be dispersed into fine particles before hydraulic grading can separate it from the sand grains.

On the Canal Zone, the first gravity rapid-sand plant constructed had a backwash rate of 12-inch vertical rise per minute. This rate was obviously too slow; and the second plant was designed with an 18-inch rise per minute. The results were better in this case, but continual handwashing had to be resorted to to prevent the growth of hard spots beginning at the side walls. The next plant built used a vertical rise of 24 inches per minute. This progression recapitulates the tendency throughout the United States during the past twenty-five years; but even the highest rates of backwash have not been entirely successful. The surface wash is therefore an obvious need.

There are several ways of accomplishing the desired end of complete filter cleansing. One is to break up the crust by tamping the bed lightly just before backwashing. In plants where filters need backwashing only once or twice a week, this is no great hardship. Another method is to spray the surface of the bed just before backwashing, using a hose and spray nozzle of convenient size and pressure. This is the method to be used at the new plant which has been designed for Springfield. A third method, which is doubtless justified where backwashing may occur at frequent intervals, is the surface wash which has been described by Hudson. Their contribution is a very practical one from the standpoint of the purification plant operator.

(Presented before the Illinois Section meeting, April 11, 1935.)

PIPE CLEANING

By Charles Haydock

(Consulting Engineer, Philadelphia, Pa.)

Pipe cleaning is not a new idea. In fact, there are English patents a century old which pertain to cleaning water mains.

The Journal of our Association has published a number of articles on the subject, the first one in 1907.

I propose to discuss pipe cleaning as one of the maintenance services available to a water works system by first presenting some thoughts on pipe lines and their carrying capacity, and following that by a brief discussion of the benefits derived from pipe cleaning by restoring the capacity and thereby improving both ordinary and fire protection service, thus improving public relations, effecting operating economies and postponing capital improvements by the more efficient use of existing facilities.

The majority of water mains are of cast iron pipe, for until recently other materials were but little used.

History establishes that, under normal conditions, cast iron pipe is a very long-lived material, but also that, in many instances, the carrying capacity of a cast iron water main is by no means as permanent as the pipe itself, for in all too many cases, tuberculation and corrosion occur quite rapidly increasing the frictional resistance to the flow of water through the main, and consequently reducing the capacity.

This reduction in carrying capacity due to tuberculation and corrosion is well known and has been discussed on many occasions. Two recent articles from The Journal are as follows:

Charles B. Burdick of Alvord, Burdick and Howson, Engineers, Chicago, Illinois, states:

Causes of Inadequacy: Inadequacies are usually caused by growth in the water consumption beyond the capacities of the pipes. This situation is aggravated from the fact that nearly all iron pipes decrease in carrying capacity as they grow older on account of tuberculation of the interior. According to the well known flow tables of Williams and Hazen, the relation between the age

of iron pipe and its capacity to deliver water under average conditions, is as follows:

Age years	*Delivery percent*
New pipe	130
5	120
11	110
19	100
28	90
41	80

Admittedly, these figures are no more than rough averages because much depends upon the quality of the water and the nature of the interior coating. The above values are supposed to apply to cast iron pipe as ordinarily coated and laid up to the year 1920 and before.

Remedies for Inadequacy: The remedies for excessive pressure losses include larger arteries at the places where most needed to reduce the distribution losses, the cleaning of pipe interior by removal of the tubercles thus restoring the interior to a smooth condition, and the actual reduction of the rates of water delivery by the introduction of elevated storage properly located with reference to the source of supply and the locality of use.

Robert A. Duff, Engineer and Superintendent of Medford Water Commission, Medford, Oregon, states:

In a city with a highly corrosive water it is advisable and economical at times to have mains cleaned. This practise is becoming increasingly more common. Often, as a result of these cleanings, such an increase in capacity is experienced that the extension of new or larger mains is forestalled.

All waters do not corrode and tuberculate pipe lines, but the waters which are the most aggressive in their attack are, in general, the soft waters, low in total solids, which are most desirable for domestic and industrial purposes.

The causes of tuberculation and corrosion are frequently discussed, but in this paper we are more concerned with methods of counteracting their effect and restoring the carrying capacity of mains which have already been reduced by tuberculation and corrosion.

Water mains are expensive. When they are installed, it is customary to construct a larger line than is immediately required, thus providing a margin for future growth. Unless a highly resistant lining is used, the carrying capacity of the pipe line usually begins to decline as soon as the line is placed in service, while if the community grows, the demands on the pipe line begin to increase at the same time. These two factors are thus working together and against the water works system. As a result, the full capacity of the line is required much sooner than anticipated. If loss of capacity and increase in demand continue, the water system is soon con-

fronted with increased operating costs due to the higher pumping head, or the necessity of taking some action to meet the changed conditions.

As Mr. Burdick stated, the remedies available are to clean the mains, provide local storage to reduce the rate of flow, or to lay new mains, and as Mr. Duff stated, the practise of cleaning mains is becoming more common.

Modern water works practise requires the use of improved linings for pipe lines which carry aggressive waters. In fact, it is usually good practise to use improved linings in all cases, because the quality of the water supply may change.

Another modern development is the treatment of water supplies to reduce their aggressiveness.

Ordinarily, pipe cleaning is undertaken to remove the tuberculation and corrosion, which normally result from the action of water on iron, although in some cases, mains have been cleaned to remove other deposits.

Everyone has heard of the strange, foreign objects which from time to time are removed from pipe lines, such as picks, shovels, axes, caulking tools, dinner buckets, overalls, shoes, hats, lumber, stones, tampers, etc. I know of a railroad tie having been removed from an 18-inch main. Difficulties had been experienced with the dam supplying this line, and no doubt the tie was carried into the pipe line by the high water which occurred at that time.

After tuberculation has occurred, it can be removed by chemical or mechanical processes. Chemical processes have been used in certain cases, but not very extensively. Recently, an English process has been advertised as having been used to clean municipal water mains in England, but it does not appear to have been used in America.

For practical purposes, pipe cleaning can be considered as a mechanical process to remove foreign material from a pipe line by passing a pipe cleaning device or machine through the pipe line.

In many instances, it is difficult directly to observe the necessity for, or the benefits to be obtained from, pipe cleaning, and equally difficult directly to observe the benefits or results of cleaning. Usually, both must be observed indirectly. It is seldom possible to observe the condition of a pipe line at more than a few points, and it is most difficult to judge conditions and coefficients by a few such observations. A pipe line may have quite a low coefficient, corresponding to the loss of a third or more of its capacity, and still the entire interior of the pipe not be entirely covered by tuberculation.

The roughness of the interior of the pipe imparts eddy currents to the water, which rapidly increase the frictional resistance to flow.

The direct benefits experienced as a result of pipe cleaning are greatest in cleaning transmission lines and pumping mains, although it may well be that greater benefit can be obtained by cleaning distribution mains, such as a circuit or a few diagonals.

It is well known that a large portion of the capacity of a water supply system is required solely for fire protection purposes. There are a number of instances of fire insurance underwriting agencies suggesting cleaning of pipe lines, in order to improve fire protection, for it is certainly true that increased capacity can usually be obtained more economically by cleaning pipe lines than by laying new mains, or installing new or additional pumps.

Public service commissions have also suggested the cleaning of mains to improve both general and public fire protection service.

Another interesting situation was where, in a bitterly contested rate case, it was contended that mains had depreciated in value to the extent of the loss in capacity sustained by the pipe lines due to the deposit of sediment from a turbid water supply, which was later filtered. In this case, there was very little tuberculation, as the water supply was not aggressive. The water company cleaned all the mains which had been laid before the construction of the filter plant, and was successful in having the mains included in the rate base at a value which reflected only the normal decrease in value due to depreciation calculated on the age-life basis.

A question frequently asked concerning pipe cleaning is,—how long do the benefits persist? This cannot be definitely answered. Certainly, the benefits cannot reasonably be expected to last as long as the original carrying capacity, for the loss in capacity indicates that the coating had already been damaged or impaired.

Lack of permanency of carrying capacity is not usually considered to be a valid objection to the use of water mains, and it is difficult to see how it can be considered a valid objection to the use of pipe cleaning as a tool to restore the impaired capacity of tuberculated mains. Pipe cleaning is not a panacea, but if the carrying capacity of a water main has been greatly reduced by tuberculation and corrosion, and this can be increased or restored by pipe cleaning, it appears logical to use pipe cleaning as a maintenance tool, for where it is practical, it usually increases the capacity of the system more economically than is possible by any other method.

Much thought has been given to the possibilities of recoating mains

in place under ground, after they have been cleaned, in an effort to make permanent, or relatively permanent, the beneficial effects of the cleaning.

Studies are being made in the United States of methods of applying a quick setting cement lining to be applied to pipes in place after cleaning, as well as mechanical applications of bituminous linings.

An English concern electrolytically deposits bituminous materials from an emulsion, but this process has not been used commercially in this country.

Pipe cleaning is not expensive in relation to the benefits derived. In fact, it is usually the cheapest way in which comparable benefits or relief can be obtained, and frequently permits the postponement of an extensive construction program, the fixed charges on which may well exceed in each year the total cost of the cleaning operation.

The length of time during which the water system enjoys the benefits of pipe cleaning depends largely on the nature of the water supply and the character of the pipe line and its protective lining or coating. Where waters are very soft, the benefits may not last very long, clearly indicating the wisdom of using better linings and treating the water to reduce its aggressiveness.

Pipe cleaning is really a service, in which the proportionate benefits depend largely on the condition of the mains to be cleaned. If the line was well laid and is in very bad condition, it may be possible to increase the capacity coefficient from 50 or 60 to 120 or 130. As the carrying capacity varies directly with the coefficient, this is truly an enormous increase.

The benefit to the water system is quite another matter. A most interesting experience was where a pump has been installed without providing for the increased frictional resistance due to the condition of the main. The shut-off head of the pump was less than the total head required to pump any material quantity of water through the line, so that while the pump could be operated, it would deliver no water until the line was cleaned. This happened some time ago, and the full details are no longer available, but as I recall it, a coefficient of about 90 was required to permit operation of the pump, while after the cleaning, the coefficient was well over a hundred.

The Consolidated Water Company of Utica, New York, has a very soft water supply, which is carried through twelve miles of 24-inch main that has been cleaned with some degree of regularity, as described by L. C. Hodge and J. W. Ackerman before the American Water Works Association. Since the last cleaning in April, 1934, the

water has been treated with chlorine and ammonia, and as a result, the value of "C" has averaged 146 in eight tests. The article in question also mentions the biological or bacterial theory of tuberculation, which was described by H. G. Reddick and S. L. Linderman before the New England Water Works Association in 1931. The results obtained at Utica are not inconsistent with a bacterial effect, the cause of which is susceptible of control by applications and ammonia.

The coefficients obtained after cleaning at Utica are quite high, and indicate that the line was well laid. Coefficients of 120 or 130 are not uncommon, after careful cleaning. On the other hand, many water systems have been much gratified by obtaining coefficients of 90 or 100, particularly where the coefficient before cleaning has been in the 50's or 60's.

Many water companies defer the cost of cleaning over the estimated life of the benefits derived. This does not decrease the cash requirements of the company, which usually pays the cost of the work at the time it is done, but it avoids a radical drop in earnings in one year, to provide increased earnings or improved operating conditions, or both, over some years in the future. In some instances, water companies have deferred the cost of an extensive pipe cleaning program over a period of as much as ten years.

The practise of deferring the cost of pipe cleaning over the expected life of the benefits is usually not so attractive to a municipally owned water system, which as a rule, is not as much interested in maintaining a consistent record of earnings, as is a privately owned system. There have been cases where municipalities have deferred the cost of cleaning, but this is a much less common practise than in the case of privately owned systems.

On the other hand, the municipal systems now have an opportunity to obtain the benefits of pipe cleaning at very small cost to themselves, which opportunity is not available to private systems. I refer to the use of the Federal and relief funds in their various forms, which are being scattered throughout the country. Many municipalities have carried out pipe cleaning programs with Federal funds, and have secured the same benefits as though they had built extensive improvements with their attendant burdens of fixed charges which will endure until the bonds have been retired, while at the same time, due to the use of labor in pipe cleaning, they have secured the benefit of relief work to the extent to which labor has been employed.

(*Presented before the Four States Section meeting, April 12, 1935.*)

COAGULATING THE WATER OF THE DELAWARE RIVER AT EASTON, PENNSYLVANIA

By R. W. Haywood, Jr.

(*Superintendent of Filtration, Lehigh Water Company, Easton, Pa.*)

The water of the Delaware River at Easton, Pa. has offered some interesting problems in its preparation for delivery and it is thought that some of them, together with their solutions, may be of sufficiently unique character, or general interest, to justify presentation before this section.

This discussion is confined to the more pertinent phases of the coagulation control at the Lehigh Water Company's filtration plant serving Easton, Pa., north of the Lehigh River, parts of the Boroughs of Wilson and West Easton, and parts of the townships of Forks and Palmer, during the calendar year of 1934.

CHARACTER OF WATER

The river water is reasonably soft, the calculated hardness varying from 1.3 grains per gallon in the spring when the normal flow is increased by melting snow and ice on the water shed, to 2.9 grains per gallon in the late fall when the flow is at its minimum. The alkalinity and pH varied throughout 1934 as shown on figure 1, the limits being a maximum of 35 p.p.m. and a minimum of 12 p.p.m. for alkalinity and a maximum of 8.4 with a minimum of 6.9 for the pH (these figures represent weekly averages). The turbidity is normally very low, exceeding 10 p.p.m. for only 12 weeks in the year. A taste which was strongly reminiscent of decaying leaves or molding wood, called the "Delaware River Taste" (1), was present in the water during the warm weather months, varying directly with temperature of the river. During the break-up of the ice in March, 1934, a decided and objectionable taste appeared, which lasted only a few days. It strongly resembled the flavor given to water in which ice containing free ammonia has been placed.

FILTRATION PLANT

The filtration plant is located on the Pennsylvania side of the Delaware River about two miles above its confluence with the Lehigh. Its intake, therefore, is above the zone of pollution of the great industrial area in the Lehigh drainage basin. It consists essentially of the primary sedimentation basin, duplicate mixing channels (operated in parallel and equipped with mechanical agitators), duplicate secondary settling basins (also operated in parallel), and eight, 1 mgd., rapid sand filters. At the maximum filtration rate of 125 mg. per acre per day, the theoretical primary settling time is one hour and the secondary settling time three hours.

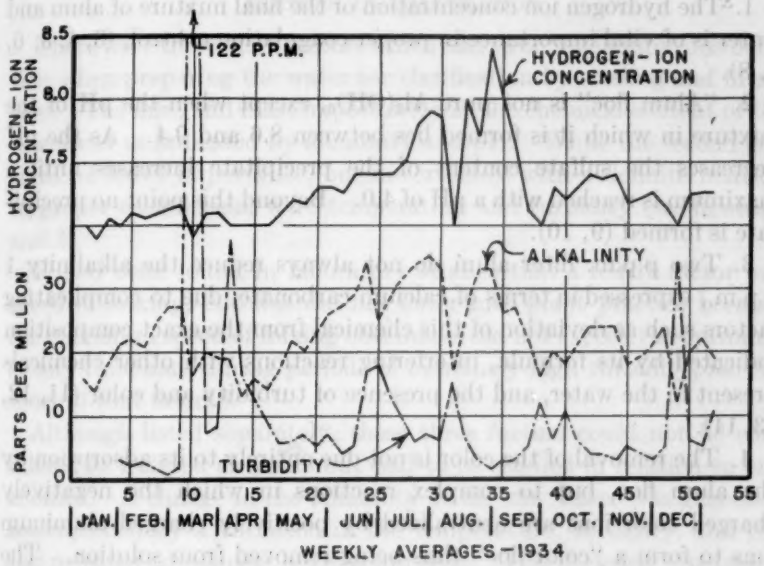


FIG. 1. SEASONAL CHANGES IN pH, ALKALINITY AND TURBIDITY OF THE DELAWARE RIVER AT EASTON, PA.

TODAY'S CONCEPTION OF COAGULATION

To review briefly, coagulation with aluminum sulfate was first thought to take place along one, of a combination, of three fundamental reactions. Assuming that all alkalinity was present as calcium, these were:

$$Al_2(SO_4)_3 + 3Ca(HCO_3) = 3CaSO_4 + Al_2(OH)_6 + 6CO_2$$
$$Al_2(SO_4)_3 + 3CaCO_3 + 3H_2O = 3CaSO_4 + Al_2(OH)_6 + 3CO_2$$
$$Al_2(SO_4)_3 + 3Ca(OH)_2 = 3CaSO_4 + Al_2(OH)_6$$

Such rules were seldom applicable in the treatment of water as was indicated by failure of solutions of alum to react according to these equations, and intensive research has changed our conception of the laws controlling coagulation materially. It is now generally accepted that:

1. The hydrogen ion concentration of the final mixture of alum and water is of vital importance in proper coagulation control, (3, 4, 5, 6, 7, 8).

2. "Alum floc" is not pure $Al_2(OH)_6$, except when the pH of the mixture in which it is formed lies between 8.6 and 9.4. As the pH decreases the sulfate content of the precipitate increases until a maximum is reached with a pH of 4.0. Beyond this point no precipitate is formed (9, 10).

3. Two p.p.m. filter alum do not always reduce the alkalinity 1 p.p.m., expressed in terms of calcium carbonate, due to complicating factors such as deviation of this chemical from the exact composition indicated by its formula, interfering reactions with other chemicals present in the water, and the presence of turbidity and color (11, 12, 13, 14).

4. The removal of the color is not due entirely to its adsorption by the alum floc, but to complex reactions in which the negatively charged color ions are neutralized by positively charged aluminum ions to form a "color floc" thus being removed from solution. The optimum hydrogen ion concentration for this reaction is below the pH range for normal formation of alum floc and varies with different waters (14, 15, 16, 17, 18, 19).

5. Controlled agitation after the addition of alum is of vital importance in floc formation (20, 21, 22).

DETERMINING COAGULATION EFFICIENCY

The final indication of proper coagulation at Easton, was the length of filter runs provided the filters were kept clean by proper washing. Due to differences in every filter, it was found unwise to decide too quickly that a short run for one filter indicated poor preparation of the water. A more accurate picture was derived from a study of average runs for three or four consecutive days, for then only could

it be said with any degree of certainty that a definite change had taken place. Fortunately, changes in the river normally occurred slowly enough to allow such a method to be used, but, in the event of rapid changes due to heavy rains, a laboratory stirring machine was indispensable and submerged lights in the mixing channels, together with floc detectors, which showed continuous samples of the influent and effluent of both channels, were a great aid in controlling coagulation.

FACTORS IN COAGULATION

There were three main factors which had to be taken into consideration when preparing the water for clarification by settling and filtration. The first, and most important, was the chemical content of the raw water as indicated by alkalinity and pH. While the second and third, of lesser importance, but nevertheless with a definite relation to proper coagulation, were temperature and turbidity (see figures 1 and 3).

It had been found in laboratory studies that a fourth factor was speed of mixing, but this did not enter into plant practice because the maximum speed of mixing obtainable did not exceed the optimum at any time and, during periods of extremely high turbidity, fell far short of that desired.

Although listed separately, these three factors could not be considered as isolated elements but had to be studied in relation to one another. A change in the alkalinity of the raw water was generally accompanied by a variation in the temperature, turbidity, and pH and required adjustment of the alum dose. This is clearly illustrated in figures 1 and 3. Usually, a change in turbidity was of no particular importance except as further indication of the necessity of changing the alum dose, for an examination of the two figures mentioned above shows that, while an increase in turbidity resulted in a corresponding increase in alum being fed for a short time, the average alum dose, based on weekly averages for 1934, varied directly as the temperature and alkalinity, the period of maximum application being the period of lowest turbidities.

However, on several instances the turbidity became a factor of paramount importance, particularly during the period of high water which accompanied the break-up of the ice in March. At this time, while perfect and quick settling floc could be obtained without difficulty on the laboratory stirrer, it was impossible to coagulate the

water in the plant sufficiently to make it filterable until high doses of alum (maximum, 230 pounds/mgm.) and primary lime (maximum, 117 pounds/mgm.) were used. Quantities of floc would form in the mixing channels almost immediately upon the addition of alum but settled out so rapidly that large quantities of suspended material remained in the supernatant. Samples of the mixing channel effluent were placed on the laboratory stirring machine and coagulated nicely, indicating that the speed of agitation in the plant was insufficient even though it was at its maximum. Only when the quantities of alum and lime were increased, and the secondary settling basins drawn down to increase the velocity through them, thus forcing floc onto the filters, was it possible to filter the water at all acceptably. During this period the controlling factor was the high turbidity, because the alkalinity was at its minimum (12 p.p.m.) for the year.

The proper control of coagulation has been found to be synonymous with efficient operation in this plant. With each additional understanding of the many factors affecting coagulation a definite benefit has been noted in the form of more positive control. For example, a study of pH variations in the river at two hour intervals during the twenty-four hours of the day, which extends from July 1 through October 31, 1934 may appear to have been only of academic interest, but was found to have tangible value in helping to explain coagulation behavior during warm weather months. It will be seen, upon examining figure 1, that the average pH of the water is not entirely a function of the alkalinity therein but tends to reach its maximum in the summer and its minimum during months of short days and weak sunlight. It is believed that this variation is a direct result of CO_2 consumption by the micro-organisms which thrive in the clear water (1).

AFFECT OF MICRO-ORGANISMS ON COAGULATION

This elevation of pH by CO_2 removal is shown in more detail in figure 2, which depicts the rise and fall of the temperature and pH of the river water and temperature of the air for two periods. The first is for one week, July 21–26, and represents the changes during hot, clear, weather. The second period is for two weeks, September 29 to October 10, and shows the changes during cold, clear weather. Assuming the temperature of the air to be an indication of sunlight, it can be clearly seen that the pH of the raw water changed more rapidly when conditions were at their best for algae growth, namely,

high temperatures and ample sunlight. This fluctuation caused some concern during the first months of operation, it being quite

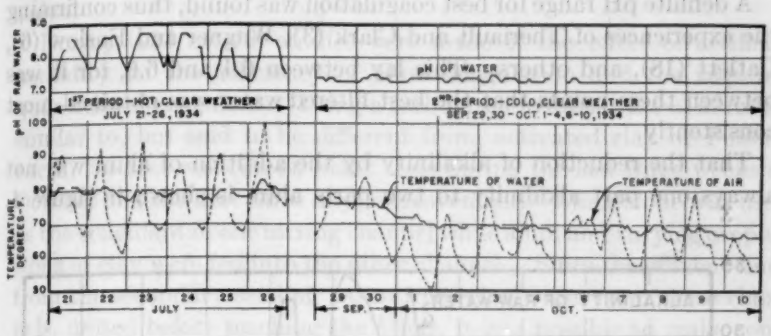


FIG. 2. TWO-HOURLY VARIATIONS OF pH AND TEMPERATURE OF THE DELAWARE RIVER AND TEMPERATURE OF AIR AT EASTON, PA.

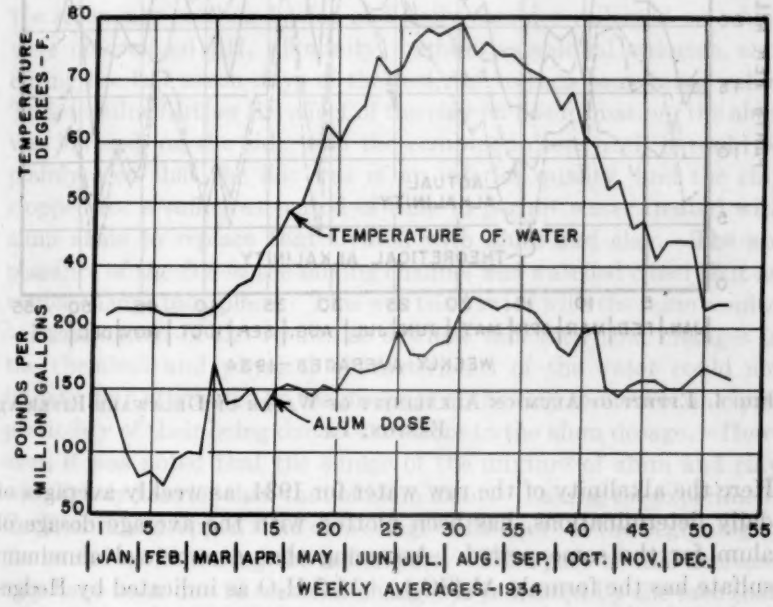

TEMPERATURE DEGREES - F.
POUNDS PER MILLION GALLONS
TEMPERATURE OF WATER
ALUM DOSE
JAN. FEB. MAR APR. MAY JUN. JUL. AUG. SEP. OCT. NOV. DEC.
WEEKLY AVERAGES-1934

FIG. 3. SEASONAL CHANGES IN TEMPERATURE AND ALUM DOSAGE—DELAWARE RIVER AT EASTON, PA.

startling to observe the pH to move from 7.2 to 8.7 within a few hours, but it was soon found that the coagulation requirements for a

given day were not measurably affected by the pH variation, probably because of low concentration of buffer salts.

A definite pH range for best coagulation was found, thus confirming the experiences of Theriault and Clark (3), Wagner and Enslow (6), Catlett (18), and others. This lay between 6.4 and 6.6, for it was between these points that the best filtered water was obtained most consistently.

That the reduction of alkalinity by the addition of alum was not always one part alkalinity to two parts alum is shown in figure 4.

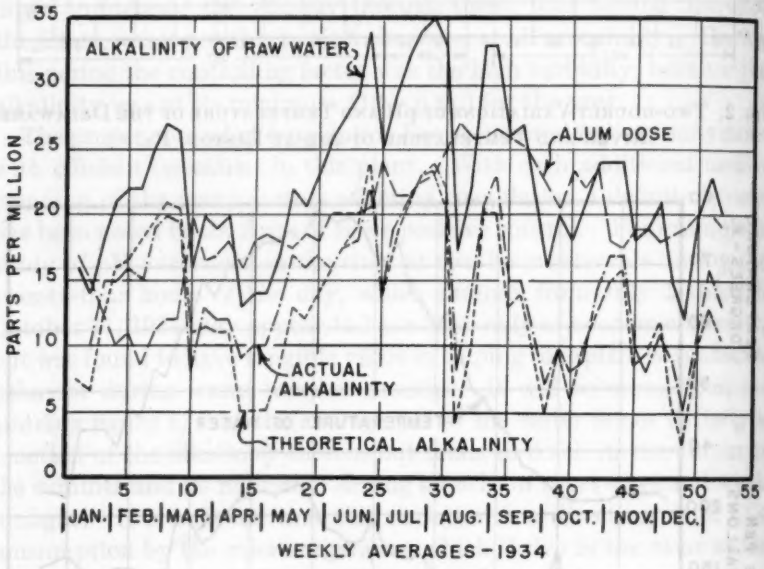


FIG. 4. EFFECT OF ALUM ON ALKALINITY OF WATER OF DELAWARE RIVER AT EASTON, PA.

Here the alkalinity of the raw water for 1934, as weekly averages of daily determinations, has been plotted with the average dosage of alum for the same period. Assuming that commercial aluminum sulfate has the formula $Al_2(SO_4)_3$. $14.2\ H_2O$ as indicated by Hedgepeth (24), the resulting theoretical alkalinity was plotted together with the actual alkalinity obtained and only in a few instances were they identical. Such results did not seem to be out of line with the experiences of Whipple (11), Ellms (12), Stein (13), Pirnie (14), and others. It is believed that a careful study of the organic load as indicated by color determinations and, possibly, oxygen consumed

values, will throw light on these differences, but, unfortunately, the number of determinations made so far has been too small to be of any value.

During 1934 careful studies were made of the effect of artificial turbidity on the efficiency of alum coagulation and the required quantities of coagulant. Using a carbonized hydrosilicate of alumina similar to, but said to be different from, activated clay or Fuller's earth, the test was run on plant scale during the last three weeks of October and the first three days of November. Alum alone was used as the coagulant in one mixing channel while alum and varying proportions of clay were fed into the other channel. Since the effluent lines from the secondary settling basins, receiving water from these channels, united before reaching the filters, it was possible to make comparative determinations only on the coagulated and settled water before it left the basins. Sampling points were established at the influent and effluent ends of both channels and at the effluent end of the secondary settling basins and daily samples collected on which were determined pH, alkalinity, turbidity, residual chlorine, and, during the last seven days of the test, the oxygen consuming value. To determine further the effect of the clay on floc formation, the alum was lowered, on the side with the combined dose, until it could be plainly seen that the floc was of an inferior quality, and the clay stopped for a sufficient period of time to permit water treated with alum alone to replace that treated with alum and clay. The appearance of the floc in the mixing channel was watched closely but no visible change took place. This was tried twice with the same results.

Examination of the results as a whole indicated that changes in the chemical and physical characteristics of the water could not be definitely attributed to the presence of the clay in view of an equal possibility of their being due to variations in the alum dosage. However, it was noted that the sludge of the mixture of alum and clay was slightly heavier than that of alum alone. During the experiment the filter runs dropped from an average of 60 hours at the beginning of the period to 24 during the latter part. That part of this decrease may have been due to seasonal changes is indicated by the fact that it was not possible to increase the filter runs above 40 hours later when effects of the clay could not possibly be considered.

SUMMARY

A review of experiences in coagulating the water of the Delaware River, Easton, Pa., during 1934 brings out several facts.

1. Improvement in control of coagulation resulted in improvement in plant operation as a whole.

2. Flocculation conformed with the rules controlling coagulation which are generally accepted today.

3. Wide variations of the pH of the river water during the warm weather months were apparently due to the presence of micro-organisms, but had no measurable effect on coagulation requirements for any given day.

4. The three main factors in coagulation were alkalinity, turbidity, and organic load and are stated in the order of their importance.

5. Careful studies were made of the effect on coagulation of the addition of artificial turbidity in the form of carbonized clay, but the results did not indicate that such treatment would be profitable.

REFERENCES

1. Thomas and Burger: Jour. A. W. W. A., 1934, 26, pp. 120.
2. Haywood: Water Works & Sewerage, October 1933, p. 349–53.
3. Theriault and Clark: Public Health Reports, Feb. 2, 1923, 38: 5. An Experimental Study of the Relation of Hydrogen Ion Concentration to the Formation of Floc in Alum Solutions.
4. Massink and Heymann: Jour. A. W. W. A., 1921, 8, pp. 239–269.
5. Greenfield: Jour. A. W. W. A., 1921, 8, pp. 397–401.
6. Wagner and Enslow: Jour. A. W. W. A., 1922, 9, pp. 373–391.
7. Baylis: Jour. A. W. W. A., 1923, 10, pp. 365–392.
8. Mortensen: Jour. A. W. W., A. 1925, 13, pp. 33–37.
9. Miller: Reprint No. 862 from the U. S. Public Health Reports, Aug. 31, 1923. On the Composition of the Precipitate from Partially Alkalinized Alum Solutions.
10. Hopkins: Jour. A. W. W. A., 1924, 12, pp. 405–509.
11. Whipple: Proc. A. W. W. A., 1911.
12. Ellms: Water Purification, 1917, 1st Ed., pp. 344.
13. Stein: Water Purification Plants, 1919, 2nd Ed., pp. 158.
14. Pirnie: Jour. A. W. W. A., 1922, 9, pp. 247–273.
15. Stein: Jour. A. W. W. A., 1921, 8, pp. 571–582.
16. Weston, Jour. A. W. W. A., 1928, 19, pp. 416–427.
17. Ellms: Water Purification: 1928, 2nd Ed., pp. 426–435.
18. Catlett: Jour. A. W. W. A., 1924, 11, pp. 887.
19. Norcom: Jour. A. W. W. A., 1924, 11, pp. 96.
20. Hopkins: Water Purification Control: 1932, pp. 5.
21. Leipold: Jour. A. W. W. A., 1934, 26: 8, pp. 1070–1084.
22. Willcomb: Jour. A. W. W. A., 1932, 24: 9, pp. 1416–1441.
23. Hedgepeth: Jour. A. W. W. A., 1934, 26: 9, pp. 1222–1233.

(*Presented before the Four States Section meeting, April 12, 1935.*)

REPORT OF COMMITTEE ON ELECTROLYSIS AND ELECTRICAL INTERFERENCE[1]

Fifteen years or so ago, when the American Water Works Association was first requested to sanction protective ground connections on water pipes, there was considerable opposition to electrical interconnections between water pipe systems and electrical distribution networks. The electrical interests, however, stressed the necessity of such ground connections as the only practical means of protecting life and property against high voltage hazards and consistently and persistently argued that such protective inter-connections would not entail any objectionable hazard, inconvenience or difficulty as far as the water works operators were concerned, because under normal operating conditions there would be no electric current flow on these connections.

On this basis and on this basis only, the American Water Works Association at its 1920 convention approved the connecting of protective grounds from the secondary systems of lighting transformers to water pipes.

The text of the 1920 resolution follows:

"WHEREAS, the grounding of the secondaries of lighting transformers to water pipes promotes safeguarding of life and does not constitute a hazard to the piping system,

"*Be It Resolved* that the American Water Works Association approves the practice of grounding the secondaries of lighting transformers to the water pipes, *providing* that the grounding is done in accordance with the rules of the National Electrical Code and the National Safety Code."

This more or less unrestricted endorsement of protective ground connections led to the installation of all sorts of promiscuous and indiscriminate ground connections between electrical systems and water pipes. In fact, certain electrical interests assumed that this sanction of protective grounds gave them an unconditional license to use the water pipes as an essential current carrying part of their electrical circuits and proceeded to do so. Trouble of various kinds soon

[1] Presented before the Annual Convention, May 6, 1935.

developed and over a period of years a steadily increasing number of complaints of sparking, electric shock and other hazards was recorded by water works operators and attributed to the establishment of electrical grounds on the pipes. In addition, there was a growing realization and fear that existing trends in the electrical industry were toward the use of types of wiring and systems of distribution which necessarily increase stray current hazards to water pipes, so that the 1927 convention of this Association deemed it necessary to modify the 1920 resolution to the extent that while still recognizing water pipe ground connections for protective purposes, it definitely specified that such ground connections must not carry electric current under normal conditions.

The text of the 1927 resolution is as follows:

"WHEREAS, this Association in 1920 endorsed the grounding of the secondaries of lighting transformers on water pipes as promoting the safeguarding of life and property without hazard to the pipe systems; and

"WHEREAS, experience with certain grounding practices, and certain trends in electrical distribution methods has shown that hazards to pipe structures and to water works employees have been and will be set up; and

"WHEREAS, certain trends in water works construction methods may seriously affect the conductivity of the water piping systems;

"*Be It Resolved*, that the American Water Works Association in convention assembled, hereby modifies its 1920 endorsement of the grounding of the secondaries of lighting transformers as follows:

"(1) The American Water Works Association approves the practice of grounding the secondaries of lighting transformers on water pipes for the purpose of safeguarding life and property, provided that appreciable electric current flows over such ground connections only during comparatively short and infrequent intervals when the ground connections are fulfilling their specific protective purposes, and provided that such ground connections impose no responsibility upon the pipe owning company.

"(2) The American Water Works Association is opposed to the use of water pipes as electrical conductors, except as noted above, and since experience with certain power distribution practices which have come into use has shown that grounding may result, and in many cases has resulted in hazard to the pipe structures and water works employees, it hereby withdraws its former general endorsement of grounding on water pipes."

In accordance with this resolution and at the insistence of representatives of the American Water Works Association, Article 9 of the National Electrical Code was modified so that the 1930 Edition of this Code recognized the existence of objectionable current flow over some water pipe ground connections and provided for the elimination of current carrying ground connections and the adoption of

alternative grounds, etc., in cases where such objectionable current flow occurred.

These provisions of the Code have not been complied with, however, and even in special cases where objectionable stray electric current has been found to be a definite and serious hazard to the pipes, it has been found impracticable to secure the removal of the current carrying ground connections, because of the apparent lack of definite responsibility for ground connections. Generally speaking, the individual property owner who knows least about the grounding problem appears to be charged with the responsibility of having ground connections installed originally to a water pipe system if one is available, before the electric company will grant him electric service. The electric company, however, accepts no responsibility for the effectiveness or integrity of such ground connections, and will not remove those found to be carrying objectionable currents. The only remedy for such cases seems to be a direct request to individual property owners to remove the connection which carries objectionable current and substitute for it some other alternative ground connection. This results in a bad customer reaction, because the property owner is asked to stand the expense of supplementary changes after having initially paid for an approved installation. In addition, the adequacy or safety of the alternative method used is necessarily an open question, especially so to the property owner who has little or no detailed knowledge of the grounding problem.

Today, where adequate studies have been made, the existence of substantial stray electric currents on many water pipe ground connections even under normal operating conditions is admitted. In fact at this year's meeting of the Electrical Committee of the National Fire Protection Association, which committee prepares and issues the National Electrical Code, the representatives of the electric light and power group used the present existence of these large stray currents on water pipe ground connections as their main arguments for the adoption of a form of interior wiring which inherently makes house piping and other house metal work systems a part of the electrical distribution system.

This form of wiring known as "bare neutral interior wiring" involves the use of one or more insulated conductors and a bare or uninsulated neutral. To prevent electric sparking and heating at points of accidental contact between this bare neutral wire and electric conduit, house piping and other metallic structures, direct

metallic connection is advocated at accessible points. The house piping, conduits and other metallic structures are therefore electrically in parallel with the neutral conductor and may carry a substantial part of any current flowing in the neutral. An investigation recently made on bare neutral installations by Underwriters' Laboratories, Inc., at the request of the Edison Electric Institute, shows in 65 per cent of the tests reported that the house piping and other metallic structures carry from 25 to 100 per cent of the total neutral current. The hazards which this current may entail are worthy of serious consideration.

Definite opposition to bare neutral wiring was expressed by a subcommittee reporting on this subject to the Electrical Committee of the National Fire Protection Association which prepares and sponsors the National Electrical Code. The recommendations of the subcommittee were overridden by the Electrical Committee and the National Electrical Code was modified to permit further extension of the use of bare neutral in range and hot water heater circuits. This change in code was very objectionable because such circuits carry much heavier electrical loads than ordinary house circuits. In the case of water heaters the bare neutral would probably be metallically connected to the water pipes at the heater. Subsequently, however, the Electrical Committee in a reconsideration of Article 5 on Wiring Methods decided not to permit the use of bare neutral for hot water heater circuits and to restrict its use to range circuits run with approved cable having a non-metallic outer covering. This therefore means that the neutral of these range circuits will have an outer covering of non-conducting braid only, whereas in the approved insulated house wiring system, both the line conductor and the neutral must have a specified thickness of rubber insulation plus an outer braid.

Your Committee was represented on the Bare Neutral Sub-Committee and on the Grounding Sub-Committee (Article 9) but has no representation on the Electrical Committee itself. Representatives of the electric light and power group and of the electrical manufacturers' group constituted sufficient strength to adopt the changes in the Code above mentioned.

The change in the National Electrical Code which grants a further extension of the use of bare neutral is probably only a preliminary step toward an attempt on the part of the representatives of the electric light and power group to utilize further the electrical conductivity

of pipes and all metallic structures for the purpose of electrical distribution.

Aggressive action by the owners and operators of water works systems in coöperation with all other interested parties is very essential. Your Committee wishes to call attention to the resolutions opposing bare neutral which have been adopted by the National Association of Master Plumbers and the American Society of Sanitary Engineering. These resolutions were presented to the Electrical Committee, but were given scant consideration.

In view of our inability to secure recognition of the basic limitations placed upon water pipe ground connections by water works operators and the many and various troubles, hazards, and complications which stray electric current flow on water pipe systems may involve, your Committee on Electrolysis and Electrical Interference believes that the American Water Works Association should protest against the present abuse of the protective grounding privilege by rescinding all general sanction of ground connections from electric light or power systems to water pipes, whether protective or otherwise, and by notifying water works operators generally that, because of the life and property hazards which may be involved in current carrying ground connections, individual water works operators who may elect to permit ground connections from electric light and power systems to the water pipes for protective purposes, do so only under the most rigid restrictions and supervision and with the question of responsibility for such ground connections definitely fixed in proper legal form.

In locations where bare neutral interior wiring is installed or contemplated either on a trial or permanent basis it is recommended that the owner or operator of the water works system should prohibit the attachment of ground connections to the water pipes from electric light and power systems, and insist upon the installation of an approved type of electrical insulating joint in the water service pipe at some point between the water meter and the street curb.

Before presenting a resolution which the Committee is to propose, it wishes to call attention to an act passed by the Legislature of the State of Washington, Chapter 169, Laws 1935, Preamble and Section 1 of which read as follows:

"Electrical Wiring.

Preamble. "An Act relating to, and prescribing the manner of installation of electrical wires and equipment, regulating sales thereof providing for the

licensing and bonding of those engaged therein, prescribing the powers and duties of certain officials in connection therewith, providing penalties, and making an appropriation, and repealing Sections 8307, 8308, 8309, 8310, 8311, and 1812 of Remington's Revised Statutes and Laws of 1919.

"Be it enacted by the Legislature of the State of Washington:

"Section 1. From and after the taking effect of this act all wires and equipment, and installation thereof, to convey electric current and installations of apparatus to be operated by said current, in, on, or about buildings or structures, for telephone and telegraph wires and equipment, and except further for signalling wires which signalling wires operate at 50 volts or less or utilize 50 watts or less, shall be in strict conformity with the provisions of this act, the statutes of the state of Washington, the rules and regulations issued by the department of labor and industries under the authority of the state statutes, and shall be in conformity with the most approved methods of construction for safety to life and property. The regulations as laid down in the National Electrical Code, as approved by the American Standards Association, and in the National Electrical Safety Code, as approved by the American Standards Association, and other installation and safety regulations approved by the American Standards Association shall be prima facie evidence of such most approved methods; and all materials, devices, appliances and equipment used in such installation shall be of a type which shall conform to any applicable standards established by the National Bureau of Standards of the U. S. Department of Commerce, and/or to the standards of the Underwriters' Laboratories, Inc.; and/or other equivalently recognized authority: *Provided*, That this act shall not limit the authority or power of any city or town to enact and enforce under power and authority given by law, any ordinance, rule or regulation requiring a higher and better standard of construction and/or a higher or better standard of materials, devices, appliances and equipment than that required by this act, but in such city or town having such higher and/or better standard such installations and materials, devices, appliances and equipment shall be in accordance with the ordinance, rule or regulation of such city or town: *Provided*, That nothing in this act shall be construed as requiring or permitting the connection of any conductor of any electric circuit with a pipe, which is connected with or designed to be connected with a waterworks piping system, without the consent of the person or persons legally responsible for the operation and maintenance of such waterworks piping system."

This Act meets the requirements of the situation very well and waterworks operators throughout the country should endeavor to secure ordinances in the towns which they serve for the purpose of preventing the connection of any conductor or any electrical circuit with a pipe which is connected with or designed to be connected with a waterworks piping system without consent of the waterworks department.

It is further recommended to the Association that the owner or operator of any waterworks system consider the requirement that

such protective ground wires be equipped with suitable isolating gaps to prevent any interchange of stray current over the ground wire under normal conditions or equipped with a suitable circuit breaker and relay control which will positively disconnect the affected electric light or power service to the building upon the passage of a relatively small amount of current over the ground wire.

Isolating spark gaps are a regular feature of standard telephone line protector equipment, so water works operators need have no hesitancy about permitting the attachment of telephone protector grounds to water pipes, because under normal operating conditions, such grounds cannot cause objectionable stray current interchange over the pipes.

Your committee on Electrolysis and Electrical Interference, with the approval of your Board of Directors, recommends the adoption of the following resolution:[2]

"WHEREAS, the American Water Works Association in 1920 sanctioned the attachment of protective ground connections from electrical supply systems to water pipes in the interests of public safety upon the theory that these ground connections would not transmit stray electric current under normal operating conditions on the electrical systems and would, therefore, not be detrimental to the water piping system or hazardous to waterworks customers or employees, and

"WHEREAS, due to changes in electrical distribution, water pipe ground connections carry substantial amounts of stray electric current normally and excessive amounts of stray current at times, so that the water piping system becomes a current carrying part of the electrical distribution system, and

"WHEREAS, these stray currents trespassing on water pipes are decidedly objectionable to the operators of waterworks pipe systems, because they constitute a hazard to the maintenance of this piping system as well as to the life and property of waterworks customers and employees, and

"WHEREAS, at the present time there is a tendency toward the adoption of a system of wiring and electrical distribution which will increase the use of the electrical conductivity of house pipes and metal work as well as street mains and make them a necessary adjunct of the electrical distribution system, thereby reducing investment in copper conductors, and

"WHEREAS, the water works operators have as their primary duty the furnishing of a safe and continuous supply of water, and since the presence of stray electric current on water pipes, or the use of water pipe systems as current carrying parts of electrical distribution systems, may seriously interfere with the primary function of the water works system;

---

[2] Ratified by the Association at the Cincinnati Annual Convention, May 6, 1935.—EDITOR.

"*Therefore Be It Resolved*, that the American Water Works Association in convention assembled hereby revokes and rescinds any and all previous approval or sanction of the practice of grounding electric light and power circuits on water pipes for protective or other purposes, and

"*Be It Further Resolved*, that copies of this resolution be sent to Mr. A. R. Small, Chairman of the Electrical Committee of the National Fire Protection Association and to Mr. H. S. Warren, Chairman of the Article 9 Sub-Committee of the Electrical Committee, which committees prepare the Grounding rules of the National Electrical Code."

Respectfully submitted,
Committee on Electrolysis and Electrical Interference,
NICHOLAS S. HILL, JR., *Chairman*
EDWARD E. MINOR
CHARLES F. MEYERHERM, *Secretary*

REPORT OF THE COMMITTEE ON DISTRICT MEETINGS OF THE INDIANA SECTION[1]

The officers of the Indiana Section and the personnel of the State Division of Public Health have felt for some time that there was need for more intimate contact with the men in charge of water plant operations in Indiana than was afforded by the annual two day meetings of the section. Under the presidency of Mr. C. C. Foutz, of LaPorte, a committee was formed to formulate plans for filling this need. This committee was known as the Committee on Water Plant Operator's Schools. Mr. L. A. Geupel, while chief sanitary engineer of the Division of Public Health, was the leading spirit in this work, and was the first chairman of the committee.

At first the committee hoped to hold district meetings in various sections of the state during the fall of 1934. However, because of the municipal elections in all cities and towns of Indiana, this plan was abandoned and the work was postponed until this spring, by which time all changes in water department personnel, resulting from the elections, would have been made.

To replace these proposed district meetings, the committee, with the help of the Division of Public Health and the Engineering Extension Department of Purdue University, conducted a conference on water purification on the afternoon of the day preceding the annual meeting of the Indiana Section. It was held at the filter plant of the Indianapolis Water Company, and was splendidly attended, and the

[1] Any person who is widely acquainted with the activities of the American Water Works Association and the relation to the Sections of the national body, realizes that as the efficiency and liveliness of interest in section activities grow, so also does the American Water Works Association grow. As the sections become more efficient in reaching and interesting water works men within their geographical boundaries, the value of the national organization to the men in the field is materially increased. In this report of district meetings carried on under the joint auspices of the Division of Public Health of the State of Indiana, and the Indiana Section of the American Water Works Association, one finds a brief record of a substantial contribution toward improvement in water supply practice, one which is worthy of emulation in other parts of the country.—Harry E. Jordan, Retiring President, The American Water Works Association.

program contained some excellent papers. It was felt, however, that it did not fill the need for which the committee was first created. Therefore, the present committee was appointed and plans were formulated for district meetings.

DISTRICTS

At its first meeting, the committee divided the state into eight districts, as shown on the accompanying map. In forming the districts the committee had two purposes in mind. The first was to build the districts around towns where men were located and employed in the water works field who would be able and willing to take charge of the meetings in their districts. The other was to keep the districts small enough so that those who would attend could drive to the meeting in the morning and return home at night, thus reducing the expense of attending to a minimum. After some changes, the following cities were selected in which to hold meetings:

Anderson, Lafayette, Terre Haute, Evansville, Batesville, South Bend and New Albany.

PUBLICITY

Our first step in advertising these district meetings was to ask the secretary of the section to send a circular letter to each water supply head in the state, explaining the nature and purpose of these district meetings and urging attendance by as many as possible from each water department. A district map was included in these letters. The letters were signed by the president of the section. Each district leader was supplied with a list of the water supplies in his district and about ten days before the meeting in his district, each leader sent out letters of invitation, enclosing programs, to each water supply in the district. A few days later, the Division of Public Health sent a letter to each department head in the district urging attendance at the meeting. Through the courtesy of Prof. W. A. Knapp of the Engineering Extension Department of Purdue University, each district leader was given the opportunity of sending stationery and a copy of his form letter to Prof. Knapp who had the letters mimeographed. This kindness of Professor Knapp was very much appreciated.

PROGRAMS

Generally speaking, the subjects covered by the meetings were quite similar. It was the aim of the committee to have the subjects

presented given in an elementary and easily understandable manner. Formal papers were largely replaced by round table discussions. The response to these discussions was very gratifying to the committee. Subjects covered at the meetings were:

1. Maintenance and extension of distribution systems.
2. Care and maintenance of well water supplies.
3. Pollution of well water supplies.
4. Laboratory control of well water supplies.
5. Chlorination and the care of chlorinators.
6. An explanation of reports from the State Board of Health.

Prof. W. E. Howland, of Purdue University, deserves especial credit and thanks for his splendid presentation, given at all the meetings, on the subject "Extension of Distribution Systems." The subject was illustrated with diagrammatic charts and tables and was the outstanding feature at each meeting. It is our hope to have Professor Howland publish this paper in bulletin form and distribute it to every water department in the state.

ATTENDANCE

When it is considered that the work of the committee was really an effort at pioneering in a new field, the attendance record is gratifying. In several instances the attendance was held down by adverse weather conditions, and by conflicting dates with other utility department meetings. The following table gives the attendance figures in summary form.

DISTRICT NUMBER	TOTAL ATTENDANCE	NUMBER WATER SUPPLIES IN DISTRICT	NUMBER SUPPLIES REPRESENTED	PERCENT SUPPLIES REPRESENTED
1–2	53	88	17	19
3	19	48	9	19
4	22	44	5	11
5	20	25	7	28
6	18	27	9	33
7	41	25	15	60
8	40	24	13	54
	213	281	75	27

CONCLUSIONS

1. The committee feels that the response to these informal gatherings was great enough to warrant the continuation of the work.

2. It is recommended that for next year the state be divided into four districts. This can be done without causing anyone attending to be away from home over night.

3. It is further recommended that the round table discussions be kept the predominating feature of the meetings.

4. The committee requests that the president write a letter of thanks to each district leader for the whole hearted coöperation and support given this committee.

Respectfully submitted,

PAUL C. LAUX, City Chemist, Anderson, Indiana, *Chairman.*
B. A. JEUP, Division of Public Health, Indianapolis, Ind.
PAUL KLEISER, Division of Public Health, Indianapolis, Ind.
NEIL KERSHAW, Chief Chemist, Indianapolis Water Company, Indianapolis, Ind.
R. P. WILEY, Professor of Engineering, Purdue University, Lafayette, Ind.

(*Report of committee, submitted to C. K. Calvert, President of the Indiana Section, A. W. W. A.*)

TYPHOID FEVER IN THE LARGE CITIES OF THE UNITED STATES IN 1934[1]

This report deals with the same ninety-three cities that have been discussed in the corresponding articles for the years beginning with 1930. The number of deaths from typhoid during 1934 in each city (except Scranton and Chattanooga, as explained in notes to tables 2 and 5) has been supplied by the respective health department. As the United States Bureau of the Census is convinced that the use of 1933 population estimates in calculating 1934 rates will at least prevent further distortions such as would occur if additional increments were added to population figures that are already too large, the rates in the present article are based on estimates of the population of the cities as of July 1, 1933, made by the Bureau of the Census. This is similar to our procedure of last year, when the 1932 midyear population figures were used to calculate the rates for 1933. It should be pointed out, however, that the 1933 population estimates used here are smaller (with only seven exceptions) than the 1932 estimates formerly furnished us by the Bureau of the Census (see, for example, table 11). This is due to the fact that the bureau has changed its method of estimating city populations. Whereas formerly the method of arithmetical progression was used for each city, the present method is based on the estimated increase in the United States as a whole (figured from birth, death, immigration and emigration figures), distributed to the states according to the percentage that each state's increase between 1920 and 1930 was of the national increase, and the state increase distributed to the counties, and the county increase to the cities.

[1] Reprinted from the Journal of the American Medical Association, 104, 23, June 8, 1935, page 2093.

The preceding articles in this series were published in the same Journal, May 31, 1913, p. 1702; May 9, 1914, p. 1473; April 17, 1915, p. 1322; April 22, 1916, p. 1305; March 17, 1917, p. 845; March 16, 1918, p. 777; April 5, 1919, p. 997; March 6, 1920, p. 672; March 26, 1921, p. 860; March 25, 1922, p. 890; March 10, 1923, p. 691; Feb. 2, 1924, p. 389; March 14, 1925, p. 813; March 27, 1926, p. 948; April 9, 1927, p. 1148; May 19, 1928, p. 1624; May 18, 1929, p. 1674; May 17, 1930, p. 1574; May 9, 1931, p. 1576; April 30, 1932, p. 1550; May 13, 1933, p. 1491, and May 19, 1934, p. 1677.

The problem of including in the rates for each city the typhoid deaths of nonresidents is as conspicuous as it has been for some years. In thirty-one of the ninety-three cities we are informed that one third or more of the typhoid deaths were in nonresidents; in ten of these thirty-one cities all the typhoid deaths were in nonresidents. These are indicated in table 9, which should be referred to also in studying tables 1–8.[2] Particulars as to the data that are unavailable for certain cities (noted in tables 2–8 as "incomplete data") are given

TABLE 1

Death rates of fourteen cities in New England States from typhoid per hundred thousand of population

	1934	1933	1932	1931	1926–1930	1921–1925	1916–1920	1911–1915	1906–1910
Bridgeport	0.0	0.7	0.0	0.7	0.5	2.2	4.8	5.0	10.3
Fall River	0.0*	0.0	0.0	0.0	2.2	2.3	8.5	13.4	13.5
Lowell	0.0*	1.0	2.0	1.0	2.6	2.4	5.2	10.2	13.9
Lynn	0.0	0.0	0.0	0.0	1.5	1.6	3.9	7.2	14.1
New Haven	0.0	1.2	1.2	1.2	0.6	4.4	6.8	18.2	30.8
Somerville	0.0	1.9	0.0	0.0	1.3	1.6	2.8	7.9	12.1
Springfield	0.0	0.6	1.9	2.6	0.4	2.0	4.4	17.6	19.9
Waterbury	0.0	0.0	0.0	0.0	1.2	1.0	8.0	18.8	
Worcester	0.0	0.5	1.5	0.5	1.0	2.3	3.5	5.0	11.8
Hartford	0.6	0.6	0.6	3.6	1.3	2.5	6.0	15.0	19.0
Boston	0.9	0.2	0.5	0.9	1.2	2.2	2.5	9.0	16.0
Cambridge	0.9	1.8	1.7	0.0	2.1	4.3	2.5	4.0	9.8
Providence	1.2	1.2	0.8	1.6	1.3	1.8	3.8	8.7	21.5
New Bedford	1.8*	1.8	0.0	1.8	1.5	1.7	6.0	15.0	16.1

* Rate computed from population as of April 1, 1930, as no estimate for July 1, 1933, was made by the Census Bureau.

in the report covering the year 1932 and in the footnotes to these tables in all earlier articles.

Nine of the fourteen large New England cities had no typhoid deaths in 1934 (table 1), and a tenth (Hartford) states that the only typhoid death occurring in 1934 was in a nonresident (table 9). Three cities (Fall River, Lynn and Waterbury) have had no typhoid deaths for four consecutive years—a remarkable record, unequaled

[2] The problem of the nonresident has been discussed at some length in previous reports; for example, J. A. M. A., 100, 1491, May 13, 1933 and 98, 1550, April 30, 1932.

heretofore by any of the large cities in this country. Only two cities in the group had 1934 rates of 1.0 or over, as against six in 1933, five in 1932 and six in 1931. In the past four years all but three cities (Boston, Hartford and Providence) have had at least one year with no typhoid deaths. Boston has a higher rate (0.9) in 1934 than its noteworthy low mark for 1933 (0.2). The New England cities as a whole have in 1934 the lowest group rate in the country (table 12);

TABLE 2

Death rates of eighteen cities in Middle Atlantic States from typhoid per hundred thousand of population

	1934	1933	1932	1931	1926–1930	1921–1925	1916–1920	1911–1915	1906–1910
Elizabeth	0.0	0.0	0.0	4.3	1.6	2.4	3.3	8.0	16.6
Jersey City	0.0	0.3	0.6	0.3	0.9	2.7	4.5	7.2	12.6
Reading	0.0	0.0	0.9	0.0	1.6	6.0	10.0	31.9	42.0
Rochester	0.0	0.3	0.3	0.9	1.7	2.1	2.9	9.6	12.8
Scranton	0.0†	3.4	1.4	2.1	1.8	2.4	3.8	9.3	31.5
Utica	0.0	0.0	1.9	0.0	1.1	3.9*			
Yonkers	0.0	0.0	0.7	1.4	0.5	1.7	4.8	5.0	10.3
Newark	0.2	0.4	0.9	0.2	0.9	2.3	3.3	6.8	14.6
Buffalo	0.3	0.3	1.2	0.7	2.7	3.9	8.1	15.4	22.8
Syracuse	0.5	1.8	0.5	0.5	0.8	2.3	7.7	12.3	15.6
New York	0.6	0.9	0.8	1.1	1.3	2.6	3.2	8.0	13.5
Paterson	0.7	0.0	0.7	2.9	1.0	3.3	4.1	9.1	19.3
Albany	0.8	0.8	0.8	2.3	1.8	5.6	8.0	18.6	17.4
Trenton	0.8	2.4	0.8	1.6	2.1	8.2	8.6	22.3	28.1
Philadelphia	0.9	0.6	1.3	0.9	1.1	2.2	4.9	11.2	41.7
Pittsburgh	1.5	0.1	1.3	1.2	2.4	3.9	7.7	15.9	65.0
Camden	1.7	3.3	2.5	4.2	4.4	5.9	4.9	4.5	4.0
Erie	1.7	0.9	1.7	0.8	0.9	2.3	6.9	49.0	46.6

* Incomplete data.

† Typhoid death rates for Scranton furnished by Pennsylvania Department of Health, Harrisburg.

this rate (0.53) is a shade better than any previous group rate, the next best being that of the East North Central cities in 1933 (0.55). This is the sixth year of progressive decline in the group rate for the New England cities and the third successive year when the rate has been below 1.0.

The Middle Atlantic cities (table 2) likewise for the past three years have had group rates under 1.0, with a progressive decline (table

12). Seven of the eighteen cities report the complete absence of typhoid deaths during 1934, Elizabeth for the third successive year, Reading, Utica and Yonkers for the second. Elizabeth's record is particularly striking after its high rates in 1930 and 1931 (4.4 and 4.3). Scranton, which had the highest rate of the group in 1933, had no typhoid death in 1934, its only previous clear record having been in 1925. The clear records for Jersey City and Rochester are the first in the histories of these cities. Pittsburgh's 1934 rate (1.5) is ten times higher than its conspicuously low rate for 1933; four of the ten deaths in 1934, however, were stated to be in nonresidents. The

TABLE 3

Death rates of nine cities in South Atlantic States from typhoid per hundred thousand of population

	1934	1933	1932	1931	1926–1930	1921–1925	1916–1920	1911–1915	1906–1910
Tampa	0.0	1.8	2.8	3.8	3.8	19.1	43.9*		
Baltimore	1.3	0.4	0.6	3.1	3.2	4.0	11.8	23.7	35.1
Jacksonville	1.4	1.4	2.8	3.0	4.4				
Washington	1.6	3.6	1.4	3.9	2.8	5.4	9.5	17.2	36.7
Miami	1.8	2.7	1.8	1.8	3.5				
Wilmington	1.9	1.9	0.9	1.9	3.1	4.7	25.8*	23.2*	33.0
Richmond	3.8	1.6	2.7	1.6	1.9	5.7	9.7	15.7	34.0
Atlanta	3.9	6.0	8.8	12.6	11.1	14.5	14.2	31.4	58.4
Norfolk	5.4*	3.8	0.8	5.4	2.2	2.8	8.8	21.7	42.1

* Incomplete data.

† Rate computed from population as of April 1, 1930, as no estimate for July 1, 1933, was made by the Census Bureau.

present rate is still well below the city's average for 1926–1930. The three cities besides Pittsburgh with a population of over half a million all had rates under 1.0, New York for the third successive year, Philadelphia and Buffalo for the second. Camden has its lowest rate since 1927.

One of the South Atlantic cities (Tampa) reports no typhoid death in 1934 (table 3). Atlanta continues to show remarkable improvement, its 1934 rate (3.9) being about one third of its rate for 1931 and its average for 1926–1930. For the first time since 1919, Atlanta relinquishes the lowest place in its group and, having a rate under 5.0, moves to the second rank among the cities of the country (table 9). Norfolk, which brings up the foot of the list in 1934, used to

stand quite consistently at the head but has had relatively high rates for four of the past five years. Baltimore had almost four times as many typhoid deaths in 1934 as in 1933.[3] The group rate for the South Atlantic cities (2.11) is slightly lower than for the two preceding years and is less than half of the corresponding rate for 1931 and for 1926–1930; it is better than the best group rate in the country in 1925.

A decided increase in typhoid mortality occurred in the cities of the East North Central group (table 4), with eighty-eight typhoid deaths

TABLE 4

Death rates of eighteen cities in East North Central States from typhoid per hundred thousand of population

	1934	1933	1932	1931	1926–1930	1921–1925	1916–1920	1911–1915	1906–1910
Grand Rapids	0.0	0.0	0.0	1.2	1.0	1.9	9.1	25.5	29.7
Peoria	0.0	0.9	1.8	1.8	0.2	3.7	5.7	16.4	15.7*
Milwaukee	0.2	0.3	0.0	0.3	0.8	1.6	6.5	13.6	27.0
Akron	0.4	1.1	0.4	1.5	1.5	2.4	10.6	21.0	27.7*
Chicago	0.6	0.3	0.4	0.4	0.6	1.4	2.4	8.2	15.8
Youngstown	0.6	2.3	1.1	1.7	1.1	7.2	19.2	29.5	35.1
Cleveland	0.8	0.5	0.2	3.4	1.0	2.0	4.0	10.0	15.7
Canton	0.9	0.0	0.9	1.9	1.4	3.3	8.9		
Dayton	1.0	0.5	1.0	0.4	1.9	3.3	9.3	14.8	22.5
Detroit	1.1	0.6	0.5	0.7	1.3	4.1	8.1	15.4	22.8
Indianapolis	1.1	0.5	1.6	1.6	2.7	4.6	10.3	20.5	30.4
Flint	1.2	0.0	1.8	0.0	1.6	4.6	22.7	18.8	46.9
Toledo	1.3	1.3	0.7	2.0	3.0	5.8	10.6	31.4	37.5
Cincinnati	1.5	0.9	3.0	0.4	2.5	3.2	3.4	7.8	30.1
South Bend	1.8	1.0	0.0	0.0					
Evansville	1.9	0.0	1.9	0.9	6.2	5.0	17.5	32.0	35.0
Columbus	2.0	1.7	1.7	2.4	2.1	3.5	7.1	15.8	40.0
Fort Wayne	6.7	0.0	2.5	1.7	4.2	2.9	7.3		

* Incomplete data.

as against fifty-four in 1933. For the first time in the past five years the progressive decline of the group rate has been interrupted, and, instead of having as usual the best group rate in the country, the East North Central cities in 1934 have fourth place. Even so, their rate is still under 1.0. Only two cities had clear typhoid records in 1934, as against five in 1933; and whereas thirteen of the eighteen

[3] An outbreak of typhoid with twenty-eight cases following a benefit supper was reported in J. A. M. A., September 1, 1934, p. 686.

cities had rates under 1.0 in 1933, there were only eight with such low rates in 1934. Twelve cities had higher typhoid rates in 1934 than in 1933—slight increases, to be sure, except in the case of Fort Wayne, which, after a clear record in 1933, has a rate of 6.7 for 1934. This is the highest rate in the group since 1930 and the highest for Fort Wayne since 1925. Six of the twelve cities with increases in 1934 state that one third or more of the typhoid deaths were in nonresidents. Detroit in 1934 had eighteen typhoid deaths, nine of them being in nonresidents, six of whom were performers in a traveling circus and had probably contracted the infection in some community which they had visited before coming to Detroit.[4] The

TABLE 5

Death rates of six cities in East South Central States from typhoid per hundred thousand of population

	1934	1933	1932	1931	1926–1930	1921–1925	1916–1920	1911–1915	1906–1910
Knoxville	0.9	7.1	8.0	7.3	10.7	20.8	25.3*		
Louisville	2.5	1.9	2.9	2.6	3.7	4.9	9.7	19.7	52.7
Nashville	2.6	7.6	7.6	3.2	18.2	17.8	20.7	40.2	61.2
Birmingham	5.8	4.0	2.5	3.0	8.0	10.8	31.5	41.3	41.7
Chattanooga	8.1†	3.2	8.0	1.6	8.0	18.6	27.2	35.8*	
Memphis	8.4	7.6	11.4	7.3	9.3	18.9	27.7	42.5	35.3

* Incomplete data.

† Typhoid deaths for Chattanooga furnished by the Tennessee Department of Health, Nashville.

Chicago increase from 0.3 in 1933 to 0.6 in 1934 is doubtless influenced by the great fire in the Union Stock Yards on May 19, 1934, when people drank water from contaminated sources.[5]

The six East South Central cities (table 5) had exactly the same number of typhoid deaths (sixty-one) in 1934 as in 1933. Only two cities (Knoxville and Nashville) showed decreases in 1934, but these were notably encouraging, Knoxville's rate being the lowest in the city's records (which began in 1920) and Nashville's the lowest in its history. Of the increases in the other four cities, that of Chattanooga is conspicuous, being more than double its 1933 rate and

[4] Freund, H. A.: Acute Outbreak of Typhoid, J. A. M. A., 103, 622, Aug. 25, 1934.

[5] Typhoid Outbreak Among Firemen, J. A. M. A., 103, 116, July 14, 1934.

about the same as its 1926–1930 average. For the past four years Memphis seems to have assumed the place at the foot of the list formerly occupied by Nashville.

Similarly, in the West North Central cities (table 6) in 1934 there was practically the same typhoid mortality as in 1933 (forty as against forty-one deaths); thirteen of the deaths, however, were reported as in nonresidents. Two of the nine cities (St. Paul and Wichita) report no typhoid deaths in 1934, and in two other cities (Omaha and Duluth) all the reported typhoid deaths were stated to be in nonresidents. It is the first clear typhoid year ever recorded for St. Paul, but Wichita has had a similar record three times before (1926, 1929, 1931). The only striking change among the cities of this group is the sudden explosion in Des Moines to a 1934 rate of 6.2, its highest rate since 1919.

TABLE 6

Death rates of nine cities in West North Central States from typhoid per hundred thousand of population

	1934	1933	1932	1931	1926–1930	1921–1925	1916–1929	1911–1915	1906–1910
St. Paul	0.0	1.4	0.7	1.1	1.4	3.4	3.1	9.2	12.8
Wichita	0.0	1.7	1.7	0.0	1.2	6.3			
Omaha	0.9	0.5	1.4	1.8	1.3	3.3	5.7	14.9	40.7
Duluth	1.0	1.0	1.0	1.0	1.1	1.7	4.4	19.8	45.5
Minneapolis	1.2	0.2	0.8	0.6	0.8	1.9	5.0	10.6	32.1
Kansas City, Mo.	1.4	2.4	1.4	1.5	2.8	5.7	10.6	16.2	35.6
Kansas City, Kan.	1.6	0.8	0.0	1.6	1.7	5.0	9.4	31.1	74.5*
St. Louis	1.7	2.2	1.2	2.0	2.1	3.9	6.5	12.1	14.7
Des Moines	6.2	2.0	0.0	0.0	2.4	2.2	6.4	15.9	23.7

* Incomplete data.

Like the two preceding groups, the West South Central cities (table 7) had practically the same mortality in 1934 as in 1933 (105 typhoid deaths in 1934 and 106 in 1933). Tulsa continues to have the lowest rate in the group, as it has had for every year beginning with 1930; but, instead of a clear record such as it had in 1932 and 1933, Tulsa in 1934 has a rate of 2.7. New Orleans continues to have the highest rate in the group, as it has had since 1931; its rate was also the highest in the country in 1931, 1933 and 1934 (table 9). The rate for this group of cities as a whole has been the highest in the country for the past three years.

The cities in the Mountain and Pacific states (table 8) average slightly less for 1934 than for 1933. There are in 1934, as in 1933, two cities without typhoid deaths. Seattle has never had such a

TABLE 7

Death rates of eight cities in West South Central States from typhoid per hundred thousand of population

	1934	1933	1932	1931	1926–1930	1921–1925	1916–1920	1911–1915	1906–1910
Tulsa	2.7	0.0	0.0	2.0	8.3	16.2*			
Houston	2.8	4.0	3.7	3.2	4.8	7.6	14.2	38.1	49.5*
El Paso	3.8	2.8	5.6	4.8	9.1	10.8	30.7	42.8	
Dallas	4.3	5.3	7.4	7.3	7.3	11.2	17.2		
San Antonio	4.9	4.9	3.6	4.2	4.6	9.3	23.3	29.5	35.9
Forth Worth	5.9	7.6	2.9	5.4	5.9	6.1	16.3*	11.9	27.8
Oklahoma City	5.9	3.4	3.9	5.6	7.4*				
New Orleans	8.9	9.1	8.6	13.9	9.9	11.6	17.5	20.9	35.6

* Incomplete data.

TABLE 8

Death rates of eleven cities in Mountain and Pacific States from typhoid per hundred thousand of population

	1934	1933	1932	1931	1926–1930	1921–1925	1916–1920	1911–1915	1906–1910
Seattle	0.0	0.8	1.1	0.5	2.2	2.6	2.9	5.7	25.2
Tacoma	0.0	0.9	1.8	0.9	1.8	3.7	2.9	10.4	19.0
San Francisco	0.1	0.1	1.5	1.4	2.0	2.8	4.6	13.6	26.3
Long Beach	0.6	0.6	0.0	0.0	1.1	2.1*			
Portland	0.6	0.0	0.6	1.0	2.3	3.5	4.5	10.8	23.2
Oakland	0.7	0.7	1.0	1.0	1.2	2.0	3.8	8.7	21.5
Los Angeles	1.0	0.6	0.6	0.7	1.5	3.0	3.6	10.7	19.0
San Diego	1.2	4.3	0.6	0.6	1.0	1.6	7.9	17.0	10.8
Denver	1.4	2.7	0.7	3.4	2.6	5.1	5.8	12.0	37.5
Salt Lake City	1.4	0.0	0.7	1.4	1.9	6.0	9.3	13.2	41.1
Spokane	2.6	0.9	1.7	0.8	2.2	4.4	4.9	17.1	50.3

* Incomplete data.

record before and Tacoma only once (1928). San Diego's rate, which rose suddenly in 1933, is about two-thirds lower in 1934, though still higher than the 1926–1930 average for that city. Denver, while still near the foot of the table, has almost halved its 1933 rate. Spokane,

TABLE 9

Death rates from typhoid in 1934

Honor Roll: No Typhoid Death (Twenty Three Cities)

Bridgeport	Peoria	Tacoma
Elizabeth	Reading	Tampa
Fall River	Rochester	Utica
Grand Rapids	St. Paul	Waterbury
Jersey City	Scranton	Wichita
Lowell	Seattle	Worcester
Lynn	Somerville	Yonkers
New Haven	Springfield	

First Rank: from 0.1 to 1.9 Deaths per Hundred Thousand (Fifty Cities)

San Francisco	0.1	Boston	0.9†	Denver	1.4
Milwaukee	0.2*	Cambridge	0.9	Jacksonville	1.4
Newark	0.2	Canton	0.9*	Kansas City, Mo.	1.4†
Buffalo	0.3†	Knoxville	0.9	Salt Lake City	1.4*
Akron	0.4	Omaha	0.9*	Cincinnati	1.5†
Syracuse	0.5*	Philadelphia	0.9	Pittsburgh	1.5†
Chicago	0.6	Dayton	1.0†	Kansas City, Kan.	1.6
Hartford	0.6*	Duluth	1.0*	Washington	1.6†
Long Beach	0.6*	Los Angeles	1.0†	Camden	1.7
New York	0.6	Detroit	1.1†	Erie	1.7
Portland	0.6	Indianapolis	1.1†	St. Louis	1.7†
Youngstown	0.6	Flint	1.2	Miami	1.8
Oakland	0.7	Minneapolis	1.2	New Bedford	1.8
Paterson	0.7*	Providence	1.2	South Bend	1.8
Albany	0.8	San Diego	1.2†	Evansville	1.9
Cleveland	0.8	Baltimore	1.3	Wilmington	1.9
Trenton	0.8*	Toledo	1.3†		

Second Rank: from 2.0 to 4.9 (Eleven Cities)

Columbus	2.0†	Tulsa	2.7	Atlanta	3.9†
Louisville	2.5	Houston	2.8	Dallas	4.3
Nashville	2.6†	El Paso	3.8	San Antonio	4.9
Spokane	2.6†	Richmond	3.8†		

Third Rank: from 5.0 to 8.9 (Nine Cities)

Norfolk	5.4	Oklahoma City	5.9	Chattanooga	8.1
Birmingham	5.8†	Des Moines	6.2	Memphis	8.4†
Fort Worth	5.9	Fort Wayne	6.7	New Orleans	8.9†

* All the typhoid deaths reported were stated to be in nonresidents.

† One third or more of the reported typhoid deaths were stated to be in nonresidents.

on the other hand, has its highest rate since 1926. San Francisco repeats its 1933 rate of 0.1—a remarkably low rate for a city of more than 500,000 population, equaled only once before by any of the twelve other cities of its size in the country (Pittsburgh in 1933). The nearest approaches in 1934 are Milwaukee (0.2) and Buffalo (0.3).

Of the thirteen cities in the country with more than 500,000 population, eight had 1934 typhoid rates below 1.0, and all thirteen had rates below 2.0, thus appearing in the first rank of table 9. All thirteen appeared together in the first rank for the first time in 1932 and again, with the exception of St. Louis, in 1933. Ten years ago, in 1924, however, only three of these very large cities had rates under 2.0.

TABLE 10

Number of cities with various typhoid death rates

	NO. OF CITIES	10.0 AND OVER	5.0 TO 9.9	2.0 TO 4.9	1.0 TO 1.9	0.1 TO 0.9	0.0
1916–1910	77	75	2	0	0	0	0
1911–1915	79	58	19	2	0	0	0
1916–1920	84	22	32	30	0	0	0
1921–1925	89	12	17	48	12	0	0
1926–1930	92	3	10	30	37	12	0
1930	93	2	6	30	23	22	10
1931	93	2	6	23	28	22	12
1932	93	1	7	13	29	29	14
1933	93	0	7	18	19	33	16
1934	93	0	9	11	27	23	23

In 1934 for the second successive year none of the ninety-three cities registered a typhoid mortality rate greater than 10 per hundred thousand (tables 9 and 10). Twenty-three cities had no typhoid deaths at all in 1934, the largest number yet reported with a perfect score. Nine of these were New England cities and seven Middle Atlantic. Eight of the twenty-three cities had had no typhoid deaths in 1933, five of these having had a clear record also in 1932 and three of them (all in New England) having had no typhoid death in the past four years. Eighteen of the twenty-three cities had had clear records at least once before; for two (Peoria and Yonkers) 1934 was the seventh year in their history without a typhoid death, while in Waterbury it was the ninth.

Five cities had no death from either typhoid or diphtheria during 1934: Elizabeth, Grand Rapids, New Haven, Seattle, Utica. It is the second successive year that Elizabeth has had a clear record for both diseases.

There are twenty cities in 1934 with rates of 2.0 and over (table 9,

TABLE 11

*Total typhoid rate for 78 cities, 1910–1934**

	POPULATION	TYPHOID DEATHS	TYPHOID DEATH RATE PER 100,000
1910	22,573,435	4,637	20.54
1911	23,211,341	3,950	17.02
1912	23,835,399	3,132	13.14
1913	24,457,989	3,285	13.43
1914	25,091,112	2,781	11.08
1915	25,713,346	2,434	9.47
1916	26,257,550	2,191	8.34
1917	26,865,408	2,016	7.50
1918	27,086,696†	1,824†	6.73
1919	27,735,083†	1,151†	4.15
1920	28,244,878	1,088	3.85
1921	28,859,062	1,141	3.95
1922	29,473,246	963	3.26
1923	30,087,430	950	3.16
1924	30,701,614	943	3.07
1925	31,315,598	1,079	3.44
1926	31,929,782	907	2.84
1927	32,543,966	648	1.99
1928	33,158,150	628	1.89
1929	33,772,334	537	1.59
1930	34,386,717	554	1.61
1931	35,137,915	563	1.60
1932	35,691,815	442	1.24
1933	35,691,815	423	1.18
1934	35,401,715	413	1.17‡

* The following fifteen cities are omitted from this table because data for the full period are not available: Canton, Chattanooga, Dallas, Fort Wayne, Jacksonville, Knoxville, Long Beach, Miami, Oklahoma City, South Bend, Tampa, Tulsa, Utica, Wichita, Wilmington.

† Data for Forth Worth lacking.

‡ This rate for the ninety-three cities in 1934 is 1.25 (total population 37,437,812, typhoid deaths 470), whereas the corresponding rates for 1930, 1931, 1932 and 1933 were respectively 1.64, 1.68, 1.34 and 1.24.

second and third ranks), as aginst twenty-five in 1933; but in 1934 nine have rates over 5.0, whereas in 1933 there were only seven with such high rates (table 10). The highest rate for both years is about the same (8.9 for 1934 and 9.1 for 1933). Of the twenty cities with 1934 rates of 2.0 and over, only four are Northern cities (Columbus, Des Moines, Fort Wayne and Spokane).

The total of typhoid deaths for the ninety-three cities is exactly the same in 1934 as in 1933 (470), but owing to the new method of estimating the city populations, mentioned earlier in this article, the rate for 1934 appears a shade higher (1.25) than the 1933 rate (1.24).

TABLE 12

Total typhoid death rate per 100,000 of population for 93 cities according to geographic divisions

	(1933) POPULATION	TYPHOID DEATHS		TYPHOID DEATH RATES					
		1934	1933	1934	1933	1932	1931	1926–1930	1925
New England	2,624,805	14	18	0.53	0.68	0.72	1.07	1.31	2.48
Middle Atlantic	12,952,300	82	102	0.63	0.78	0.97	1.06	1.40	2.97
South Atlantic	2,367,307	50	55	2.11	2.31	2.23	4.29	4.50	7.01*
East North Central	9,643,100	88	54	0.91	0.55	0.70	1.00	1.29†	2.32†
East South Central	1,242,600	61	61	4.91	4.91	6.20	4.09	8.31	13.00
West North Central	2,704,500	40	41	1.48	1.51	1.03	1.34	1.83	3.43
West South Central	1,934,800	105	106	5.43	5.40	5.20	6.97	7.32‡	13.08‖
Mountain and Pacific	3,968,400	30	33	0.75	0.82	0.87	1.07	1.80	2.33

* Lacks data for Jacksonville and Miami.

† Data for South Bend for 1925–1929 are not available.

‡ Lacks data for Oklahoma City in 1926.

‖ Lacks data for Oklahoma City.

For the seventy-eight cities for which we have complete data since 1910 (table 11), the 1934 total of typhoid deaths is ten less than the 1933 total, and the 1934 rate is 1.17, compared to 1.18 in 1933.

In four of the eight geographic divisions of the country (table 12) there were fewer typhoid deaths in 1934 than in 1933, and in three other divisions there were either exactly the same number as in 1933 or one less. The 63 per cent increase in typhoid deaths in the East North Central cities exactly offsets the decreases recorded for the other divisions. Even so, the different divisions have about the same relative typhoid rates in 1934 as in previous years. The cities

of four divisions—New England, Middle Atlantic, East North Central and Mountain and Pacific—record very little typhoid, all having had rates below 1.0 for the past three years and all (except for the 1934 check in the East North Central group) having had a continuous decline in rate during the same period. The West North Central group (table 6) stands alone with consistent rates between 1.0 and 1.5. The three Southern groups have their usual order in 1934: South Atlantic (rate 2.11), East South Central (4.91) and West South Central (5.43). The rates of the South Atlantic and West South Central cities have remained about level for the past three years, the level in both groups being well below their rates for 1931 and for 1926–1930. The East South Central group rates, on the other hand, have been higher in the past three years than in 1930 and 1931.

While the total number of typhoid deaths in the ninety-three large cities of the United States was the same in 1934 as in 1933, two points in the 1934 typhoid history are encouraging as to the possibility of further reduction in the urban deaths from this disease: three cities finished their fourth consecutive year without a single typhoid death; and Atlanta, whose rate had continued through 1931 at its 1916–1920 level, has consistently lowered its typhoid mortality by 30 or 35 per cent of each of the past three years.

STANDARD SPECIFICATIONS FOR RIVETED STEEL ELEVATED TANKS AND STANDPIPES[1]

The American Water Works Association recommends these specifications as a uniform guide in the preparation of the design, the definition of materials, stresses, workmanship, fabrication and methods to be followed in the erection and the construction of elevated tanks, supporting towers and standpipes. It is recognized in presenting these specifications that construction of this type must necessarily be adapted to a wide variation in conditions and that no specification can be written to cover all the details of those variations. It is hoped that the design, materials and workmanship will in all cases, be at least equal to the requirements herein set forth.

Various comments or suggestions by the Committee appear scattered through the specifications. These have been set in small type and usually preceded by the word "note." They do not form parts of the specifications.

PART 1. GENERAL SECTIONS

Definitions

SECTION 1. Where the word "elevated tank" or "tank" is used, it is understood to mean a cylindrical shelled tank supported upon an elevated structure or tower, to which it is securely anchored, so that the tank and its supporting structure act as a unit.

Where the word "tower" is used, it is understood to mean the supporting steel frame work or other structure supporting an elevated tank.

By "tower height", it is understood to mean the vertical distance measured from the top of the foundation pier or piers to the bottom

[1] The first draft of a specification for Steel Elevated Tanks and Standpipes was prepared by Sub-Committee 7-H, Louis R. Howson, Chairman, and published in THE JOURNAL in December, 1931, with request for criticism from the membership. Following receipt of criticisms and study of them by the members of the Sub-Committee this final standard has been prepared, adopted by Sub-Committee 7-H, approved by the Chairman of Committee 7 and by the Committee on Water Works Practice. It has not yet been acted upon by the Association.

This standard is intended to cover riveted work only.

capacity line (i.e. the low point of the hemispherical or ellipsoidal bottom) of the elevated tank.

Where the word "standpipe" is used int hese specifications, it is understood to mean a tank having a cylindrical shell and a flat bottom resting upon a prepared foundation located directly upon the ground. It may or may not be anchored to this foundation, depending upon its proportions.

Information to be furnished by purchaser

SECTION 2. The purchaser in asking for proposals, shall furnish the bidders with complete information as to local conditions, location and size of pipe connections, nature of soil upon which foundation is to rest, location of the site of erection with respect to railroad sidings and points at which materials can be unloaded. Purchaser shall also advise definitely the construction and materials which will be furnished to the contractor, the capacity of tank or standpipe required and minimum and maximum dimensions.

After the contract has been awarded and before any work is commenced, the contractor shall submit to the purchaser or to the purchaser's engineer for approval, duplicate prints of stress sheets and shop drawings.

Capacity

SECTION 3. The standard sizes for elevated tanks and standpipes shall be those recommended by the Division of Simplified Practice of the Department of Commerce and shall be as follows:

5,000	60,000	500,000
10,000	75,000	750,000
15,000	100,000	1,000,000
20,000	150,000	1,500,000
25,000	200,000	2,000,000
30,000	250,000	2,500,000
40,000	300,000	3,000,000
50,000	400,000	4,000,000

All in United States gallons net capacity

PART 2. ELEVATED TANKS

Design and materials

Form and proportion

SECTION 2.1. Elevated tanks shall be cylindrical in cross-section and may have a flat bottom supported on a grillage of "I" beams and concrete, but ordinarily shall have a suspended bottom of conical, spherical or ellipsoidal shape, or a supported bottom made up of a composite of the above surfaces.

Note: The total height of an elevated tank measured from the lowest point of the bottom to the overflow level should not be more than one and one-half times the diameter of the tank, except in unusual cases.

Anchorage

SECTION 2.2. The tower columns shall be attached to the foundations by means of anchorage bolts capable of resisting the greatest net uplift that can be produced by the wind load with stress in bolts not exceeding 15,000 pounds per square inch.

Towers

SECTION 2.3. The height of towers shall be that necessary to meet the requirements of elevation as requested by the purchaser in asking for bids.

No elevated tank shall have less than four columns in its supporting tower. Four column towers may be used for elevated tanks up to 150,000 gallons capacity, providing the diameter of the tank is such that the distance between columns at the top does not exceed 20 feet. For larger tanks, six or more columns shall be used.

In order to avoid unnecessary eccentric loading on the columns and local stresses in the tank bottom, the connections between the columns and the side of the tank shall be made in such a manner that the center of gravity of the column section intersects as closely as possible, the center of connection between the balcony and the sides of the tank. The columns shall be designed for any eccentricity. Enough rivets shall be provided above this intersection to transmit the total column load.

Tops of inclined columns shall be connected by a continuous, horizontal circular girder if the tank bottom is suspended. The outer flange should preferably be symmetrical about the web plate, but if eccentric, it shall be braced laterally or designed for the eccentricity. The bending moment shall be considered as carried entirely by the flanges, the shear by the web and the compression by the flanges and web in proportion to their areas at the section considered. Splices shall be designed to transmit the load in the spliced member. The horizontal component of the maximum column load shall be transmitted to the girder. The circular girder may be used as a balcony. Drain holes shall be provided in the web plate.

If the columns are vertical and the tank bottom suspended, the circular girder shall be as stated in the preceding paragraph, or else shall be a structural shape connected to the tank by the rivets of the joint between the shell and bottom, the shape extending as close as possible to the columns.

Tower columns may be vertical or inclined, as conditions may

warrant, but for aesthetic design, inclined legs are generally to be preferred, the minimum batter or inclination being $\frac{1}{24}$ of the height. Greater inclination may be desirable for high towers in order to obtain greater stability.

Bracing in the tower

SECTION 2.4. The vertical supply pipe shall be thoroughly braced laterally with adjustable diagonal bracing at the panel points of the tower.

The diagonal bracing in the tower shall be calculated for an initial stress of 3,000 pounds per square inch, in addition to wind stresses. Diagonal sway bracing shall be by riveted or adjustable rod members. Adjustable rod members may be used in tanks of less than 500,000 gallons capacity. On larger tanks, sway bracing shall be by riveted members.

Balcony

SECTION 2.5. In the case of batter post towers, provision shall be made at the junction of the tower with the tank to take care of the horizontal thrust due to the inclination of the columns. With towers over 50 feet in height, this shall be cared for by a balcony which shall be constructed about the tank at the level where the cylindrical portion joins the bottom. The balcony shall be at least 24 inches wide for tanks up to 15 feet in diameter and not less than 30 inches for tanks of larger diameters. Balcony shall have a floor plate with a minimum thickness of $\frac{5}{16}$-inch suitably punched for drainage. The balcony shall be provided with a structural steel hand railing at least 3 feet high constructed so as to stiffen the outer chord of the girder.

Riser pipe

SECTION 2.6. The supply or riser pipe shall be steel pipe having an internal diameter depending upon tank size.

Note: Ordinarily the diameter will be from 3 to 4 feet for tanks up to 150,000 gallons and from 5 feet to 10 feet for larger tanks.

The circumferential joints shall be riveted and shall be designed to withstand the compressive load imposed by the tank and the vertical joints shall be designed to safely withstand the bursting stress in the riser. The joints shall in general conform to the requirements for joints in the steel plate work of the tank. The supply pipe shall extend from the level of the top of the concrete foundations to the bottom of the tank. The lower end of the supply pipe shall be closed by means of a suitable flanged bottom plate $\frac{3}{8}$-inch thick.

The contractor shall furnish and rivet to the bottom plate of the elevated tank riser pipe, a fitting of such dimensions and construction that a — inch, class — pipe may pass through the fitting and a lead joint be poured.

The fitting shall have grooves for calking and shall be of design approved by the engineer. The contractor shall furnish a silt stop (preferably removable and suitable for setting into the inlet pipe). The silt stop shall be of cast iron or steel, set 12 inches high above the bottom of the riser pipe bottom when in place, and shall be not less than $\frac{3}{8}$-inch thick.

Just above the bottom of the supply pipe and in the side of the riser, there shall be an elliptical manhole having a nominal size of not less than 12 inches by 16 inches, fitted with a proper cover, gasket and fastening to make it water-tight. The opening shall be suitably reinforced and a suitable crane provided to hold the cover.

Note: On large riser pipes an 18-inch by 24-inch manhole is desirable.

The vault in the concrete foundation which supports the bottom of the steel supply pipe, will be built by the owner, but the contractor shall grout between the steel plate and the concrete after the tank is erected.

Note: In elevated tanks designed with riveted or welded steel riser pipe without expansion joint, the riser pipe shall be designed to carry a portion of the load of the suspended bottom to the foundation. The minimum load for the riser pipe foundation shall be considered a column of water the diameter of the riser pipe at its junction with the suspended bottom of the tank and the height of the riser at this point above the foundation, plus a cylinder of water 4 feet greater in diameter than the riser and extending from the top of the riser to the maximum elevation of water in the tank.

Top stiffeners

SECTION 2.7. On all elevated tanks not provided with a roof or adequately stiffened by the roof or cover, the top of the tank shall be stiffened with structural shapes having a section modulus which in inches shall be at least equal to $\frac{1}{250}$ part of the square of the diameter in feet. Joints in stiffener members shall be spliced. The thickness of any structural shape making up the stiffener shall be not less than $\frac{3}{8}$-inch.

Overflow

SECTION 2.8. An overflow — inches in diameter shall be provided. Discharge shall be visible and pitched to drain. Exposed length shall not exceed 4 feet. Sufficient space shall be provided between the roof and the top edge of the tank to permit of the overflow of water in case the tank is overfilled.

Roof

SECTION 2.9.

Note: All elevated tanks used for the supply of water for domestic and sanitary purposes, should be covered. With elevated tanks enclosed by masonry having a roof, no independent tank roof will be required if construction of the enclosure is such as to prevent access of birds or animals to the tank.

The roof shall provide for ventilation of the space above the water in the tank, but shall be so designed and constructed as to prevent the ingress and egress of birds, bats or similar animals.

The roof shall be either self-supporting or if the diameter and pitch require, shall be supported by suitable angle or channel rafters or by other satisfactory means. The roof plates shall have a thickness of not less than $\frac{3}{16}$-inch.

There shall be an ornamental finial on the top of the roof sufficiently strong to support a revolving ladder and a painter's hoist.

A trap door about 2 feet square shall be provided in the roof plate conveniently arranged to be operated from the ladder.

Ladder

SECTION 2.10. A steel ladder shall be provided for the elevated tank, supported by one of the tower columns and shall extend from a point 12 feet above the top of the concrete foundations to the top of the balcony rail. The side members of the ladder shall be bent over and fastened to the balcony or tank in such manner as to form suitable hand rails. On tanks provided with a roof, there shall be a ladder extending from the level of the balcony to the peak of the roof so constructed and fastened to the finial in the peak of the roof and running on rollers as to permit the ladder being swung around the tank, or a fixed ladder from the balcony to the eave and a revolving ladder from the eave to the peak with provision for attaching a boatswain's chair to the lower end. On tanks without a roof, there shall be a fixed ladder extending from the balcony to three feet above the top of the tank shell, the side members to be constructed in such manner as to form suitable hand rails. The side members of the ladder shall be not less than $\frac{3}{8}$-inch by 2 inches, spaced 15 inches apart, the rungs not less than $\frac{5}{8}$-inch spaced 12 inches apart and the brackets not less than $\frac{1}{2}$-inch by 3 inches, spaced not to exceed 12 feet apart.

Note: Vertical ladders on high tanks may be provided with safety cages consisting of 30-inch diameter, 2-inch by $\frac{3}{16}$-inch bar hoops attached to the sides of the ladder at not to exceed 5 foot intervals. Hoops should be reinforced with three 2-inch by $\frac{3}{16}$-inch vertical bars spaced at equal distances around the circumference.

Materials

SECTION 2.11. Steel for plates, rivets and rolled shapes shall conform to the "Standard Specifications for Structural Steel for Bridges", of the American Society for Testing Materials, Serial Designation A-7 Structural Steel, of the latest revision.

Steel for deformed tank bottoms shall be of pressing quality, the physical properties being the same as for rivet steel, as cited in Specifications A-7 for Structural Steel, latest revision.

Rods shall be of mild steel suitable for upsetting, welding and threading.

Copper bearing steel or other material resistant to air corrosion shall be used for plates in balcony and roof.

Copper bearing steel shall contain not less than 0.2 per cent copper. In all other respects, it shall conform to the above standard specifications.

Requirements for design

SECTION 2.12. The loads to be used for designing the tank, tower, and foundations shall be as follows:

Dead load. The dead load shall be the estimated weight of all permanent construction and fittings.

Live load. The live load shall be the weight of the liquid contents of the tank when full. If the roof has a slope of less than 25 degrees with the horizontal, the live load shall also include a snow load of 25 pounds per square foot on the horizontal projection of the roof for all tanks built in locations where snowfall occurs. The design of the tower need not include the weight of the water contained in the large steel riser which shall be considered as a cylinder extending from the water surface to the base of the riser, the diameter of the cylinder being taken as that of the riser at the foundation.

Wind load. Wind pressure shall be assumed to be 30 pounds per square foot on a vertical plane surface or a maximum of 50 pounds per lineal foot of column. In calculating the wind load on the cylindrical surface, six-tenths (0.6) of the above pressure shall be applied to the total area of the vertical projection, and the point of application of the load shall be at the center of gravity of the projected area. The load on the tower shall be assumed to be concentrated at the panel point. Wind load on the vertical projected area of spheroidal or conical surfaces shall be 15 pounds per square foot. Stresses due to wind may be neglected if they are less than 25 per cent of the dead and live load.

In locations exposed to unusual winds, the assumed wind pressure shall be taken at 40 pounds per square foot.

Balcony and walkway loads. Balcony shall be designed to carry a concentrated load of 1000 pounds at any point without overstressing.

Allowable stresses

SECTION 2.13. All parts of the structure shall be designed to resist safely the maximum stresses that can be provided by any combination of the above loads. The maximum stresses in pounds per square inch produced by the above loads shall not exceed the following:

	Pounds per square inch—net section
Tension:	
Tension in cylindrical tank plates	15,000
Tension in bottom course and formed bottom plates	12,000
Shearing:	
Rivets in cylindrical portion of tank	11,250
Rivets in bottom course and formed bottom plates	9,000
Rivets in main column connection to tank	7,500
Bearing:	
Bearing on rivets shall be twice the allowable shear.	
Compression:	
Compression in steel	18,000

For compression members, the permissible unit stress of 18,000 pounds per square inch shall be reduced by the formula:

$$p = \frac{18{,}000}{1 + \frac{L^2}{18{,}000\, r^2}}$$

where p = permissible working stress in compression in pounds per square inch; L = length of member from center to center of connections in inches; r = least radius of gyration of section in inches.

The ratio L/r shall not exceed 125 for main members and 175 for struts and roof supporting members.

All parts of the tower shall be proportioned in accordance with Section 5 of the Standard Specifications of the American Institute of Steel Construction.

Basis of stress computations

SECTION 2.14. The pressure possible at the bottom of each course in the cylindrical portion of the tank shall be assumed as constant throughout such course, and the tensile stress computed accordingly.

The tensile stress in tank bottoms shall be assumed as constant throughout each course and equal to the maximum stress intensity that may exist anywhere in such course.

Careful analysis and determination of the stresses, both tension and compression, that may exist in a suspended tank bottom, must be made and provided for under the unit stresses set forth in Section 2.13.

Net sections

SECTION 2.15. Net sections shall be used in calculating the tensile strength in plates and members. In deducting rivet holes, their diameter shall be taken $\frac{1}{8}$-inch larger than the undriven rivets for punched holes and $\frac{1}{16}$-inch larger for reamed or drilled holes.

Minimum thickness

SECTION 2.16. The minimum thickness of metal in the tank shall be $\frac{1}{4}$-inch for tanks up to and including 500,000 gallons capacity. For larger sizes, $\frac{5}{16}$-inch shall be minimum thickness.

The minimum thickness of metal in towers shall be $\frac{1}{4}$-inch except in the webs of channels and "I" beams and in minor parts which carry no load.

The minimum thickness of cover plates shall be $\frac{1}{4}$-inch but not less than $\frac{1}{50}$ of the distance between rivet lines. A cover plate may be used on but one side of a column.

Rivets

SECTION 2.17. The diameter of rivets shall be not less than the thickness of the thickest plate connected in any joint. The minimum permissible diameter of rivets shall be $\frac{5}{8}$-inch in tank plates.

The rivets used shall be of the proper length and diameter to completely fill the rivet holes. Rivets $\frac{5}{8}$-inch and over shall be hot driven and shall have a strong and satisfactory hemispherical or rounded conical head central on the shank and concentric with the rivet holes. Rivets shall have diameters within $\frac{1}{16}$-inch of the diameter of the rivet holes in which they are to be placed. Rivets $\frac{1}{2}$-inch and under may be driven cold. When so driven, the heads may be practically flat.

Spacing of rivets

SECTION 2.18. The minimum rivet pitch shall be not less than three times the nominal diameter of the rivet.

The maximum pitch along a calked edge, except for column connections, shall not exceed two and one-half times the thickness of the thinnest plate connected for single rivet joints or three times the thickness of the thinnest plate in joints having more than one row of rivets, plus, in each case, the diameter of the rivet hole in inches plus 1.5 inch.

Maximum pitch along uncalked edges shall not exceed thirty times the thickness of the thinnest plate connected.

Edge distance

SECTION 2.19. The distance between the center of the outer row of rivets and the edge of plate shall not be less than one and one-half times the diameter of the rivets. Where edges are beveled, the distance from the center of rivets to the toe of the bevel shall not exceed one and three-fourths times the diameter of the rivets.

Workmanship

Shearing

SECTION 2.20. All shearing shall be neatly and accurately done. All cuts shall be clean without drawn or ragged edges and without splitting away from the sheared edge. Calked edges may be bevel-

sheared on plates $\frac{5}{8}$-inch thick and under. Plates over $\frac{5}{8}$-inch thick shall have calking edges planed or flame cut by a mechanically guided device.

Beveling

SECTION 2.21. The edges of all plates and butt straps which are to be calked, shall be beveled to an angle of approximately 70 degrees with the plane of the plate for plates up to $\frac{1}{2}$-inch in thickness and for heavier plates, the angle shall be increased to approximately 80 degrees.

Punching, reaming and drilling

SECTION 2.22. Rivet holes in material $\frac{1}{2}$-inch thick and under may be punched or drilled full size.

Rivet holes in materials over $\frac{1}{2}$-inch to and including $\frac{3}{4}$-inch thick and in thinner plates for butt joints shall be either drilled from the solid or punched $\frac{1}{4}$-inch smaller in diameter than the nominal diameter of the rivet and then reamed to size.

Rivet holes in material over $\frac{3}{4}$-inch thick shall be drilled.

For the butt joints, rivet holes shall be laid out the required distances from the machined edges of the plates, and where the edges are beveled, the distances shall be laid out from the edge of the surface of the plate with which the rivet heads will be in contact.

The edges of the rivet holes which will be in contact with the rivet heads shall be beveled $\frac{1}{16}$-inch with a twist drill or a rose reamer.

Punched rivet holes shall be made with a punch whose diameter shall not exceed the diameter of the rivet by more than $\frac{1}{16}$-inch. The diameter of reamed and drilled holes shall not be more than $\frac{1}{16}$-inch larger than the rivet.

All punched and reamed holes shall be clean cut without torn or ragged edges. Rivet holes shall be accurately spaced. Any noticeable burrs shall be removed by a tool countersinking not more than $\frac{1}{16}$-inch.

Shaping plates and angles

SECTION 2.23. The plates, straps, angles and other structural shapes shall be accurately shaped to the forms required. All plates, butt straps and structural shapes shall be accurately cold rolled or pressed to the radius of the course for which they are intended. The curvature of any plate, angle or other structural shape shall be uniform throughout without flat places at the ends.

Scarfing

SECTION 2.24. Where three thicknesses of metal overlap, the corner or end of the interior plate shall be scarfed to a thin edge. The beveled surface shall be smooth.

Plates to be scarfed may be heated to a cherry red color, but not hot enough to ignite a piece of dry hard wood when applied to it. If

scarfed by pressure, such scarfing may be done cold. Most careful attention shall be paid to all scarfing.

Protection of steel

SECTION 2.25. All structural steel received on the work shall be immediately placed upon substantial shores or blocking of sufficient size and strength to prevent any metal from touching the ground, and in such a manner that water cannot collect thereon and so that the material will be protected against bending under its own or superimposed loads. Any steel members, which may become bent, shall be replaced by new material. Before assembling, the surfaces to be in contact with each other shall be thoroughly cleaned.

Setting columns

SECTION 2.26. The columns shall be set to exact elevation and position on wedges and while thus supported, the spaces beneath shall be thoroughly filled with Portland cement grout. The nuts on all anchor bolts shall be screwed down tight and the tops of the bolts up-set sufficiently to prevent the nuts from becoming loose.

Assembling and erecting

SECTION 2.27. All burrs, projections, dirt and rust shall be removed from faying surfaces before the plates are assembled, and the plates shall be securely bolted at the joints in advance of the riveting so as to be in close contact before the rivets are driven.

All the rivet holes in the plates shall coincide, and any holes which do not do so when plates are assembled shall be reamed, and larger rivets shall be used where necessary. The use of drift pins will be allowed only for bringing together the several parts of the structures, and force shall not be used to fair the rivet holes, under any circumstances.

If, in the opinion of the engineer, the holes in any plate cannot be made to coincide without weakening the joint or producing unsatisfactory work, the contractor shall provide a new and satisfactory plate.

The use of sledges in erecting tank structures shall be limited to the necessary laying up or forging to close the shell plates around scarfed corners or butt straps. Care shall be taken to prevent material from falling, or from being in any way subjected to heavy shocks.

Riveting

SECTION 2.28. The field formed head of rivets shall be on the same side of the tank as that on which the calking will be done. Heads shall present a uniform appearance.

The rivets shall be driven so as to pinch the plates firmly, and all loose or defective rivets or rivets with badly shaped heads or with heads out of position, shall be cut out and replaced in a satisfactory manner.

The rivets shall be driven by pressure tools wherever practicable. Pneumatic hammers shall be used in preference to hand driving.

Calking

SECTION 2.29. Tank shall be made tight by calking. No foreign materials shall be placed between the surface of overlaps.

Calking shall be done with a round nose tool and only by experienced and skilled men.

Testing

SECTION 2.30. After the structure has been erected (but prior to painting) and all seams have been calked, it shall be filled with water furnished by the owner and shall be tested for water-tightness. All leaky seams shall be calked by the contractor, to make the tank watertight. Loose rivets shall be cut out and replaced by new ones but minute leaks at tight rivets may be calked. Any defects revealed by this test shall be made good by the contractor in a satisfactory manner. Tests for water-tightness shall be repeated until the tank is perfectly tight. The contractor shall guarantee the tightness of the elevated tank.

PART 3. STANDPIPES

Design and materials

Forms and proportions

SECTION 3.1. Standpipes shall be cylindrical in cross-section and shall have a flat bottom resting directly on foundation.

Note: Attention is called to the fact that a standpipe or flat bottom tank having a vertical height much greater than the diameter of the tank shell, is not an economical form or proportion. For service purposes and maintenance of uniform pressure on the distribution system, it is always desirable to make the diameter as large as possible, thereby obtaining the desired capacity with a minimum change in elevation of water surface.

Anchorage

SECTION 3.2. The standpipe shall be attached to the foundation by means of anchorage bolts in all cases where any net uplift or overturning moment exists when conditions are such that the standpipe is empty and full wind pressure is exerted. If uplift exists, anchor bolts shall be not less than $1\frac{1}{4}$-inch in diameter set deep enough to take the uplift and of sufficient cross-section that the unit stress will not exceed 15,000 pounds per square inch under maximum conditions. Anchor bolts shall be in even multiples of the number of plates forming the lower ring of the standpipe. Anchor bolts shall be attached to the cylinder by properly designed lugs made of structural steel or bent plates. Cast iron lugs shall not be used.

Pipe connections

SECTION 3.3. The contractor shall furnish and rivet to the bottom plate of the standpipe a plate $\frac{1}{2}$ inch thick around the opening for

the inlet pipe connection and a fitting of such dimensions and construction that a — inch, class — pipe may pass through the fitting and a lead joint be poured.

The fitting shall have grooves for calking and shall be of design approved by the engineer. The contractor shall furnish a removable silt stop suitable for setting into the inlet pipe. The silt stop shall be of cast iron or steel, shall set 6 inches high above the bottom of the standpipe when in place, and shall be not less than $\frac{3}{8}$-inch thick.

Manhole

SECTION 3.4. Just above the bottom in the side of the standpipe, there shall be an elliptical manhole having a nominal size of not less than 18 inches by 24 inches, fitted with a proper cover, gasket and fastening to make it water-tight. The opening shall be suitably reinforced and a suitable crane provided to hold the cover.

The vault, built in the foundation which supports the bottom of the standpipe, will be built by the owner, but the contractor shall grout between the steel plate and the concrete after the standpipe is erected.

Top stiffeners

SECTION 3.5. On all standpipes not provided with a roof or adequately stiffened by the roof or cover, the top of the standpipe shall be stiffened with structural shapes having a section modulus which in inches shall be at least equal to $\frac{1}{250}$ part of the square of the diameter in feet. Joints in stiffener members shall be spliced. The thickness of any structural shape making up the stiffener shall be not less than $\frac{3}{8}$-inch.

Overflow

SECTION 3.6. An overflow — inches in diameter shall be provided. Discharge shall be visible and pitched to drain. Exposed length shall not exceed 4 feet. Sufficient space shall be provided between the roof and the top edge of the standpipe to permit of the overflow of water in case the standpipe is overfilled.

Roof

SECTION 3.7. All standpipes used for the supply of water for domestic and sanitary purposes, should be covered. With standpipes enclosed by masonry having a roof, no independent standpipe roof will be required if construction of the enclosure is such as to prevent access of birds or animals to the standpipe.

The roof shall provide for ventilation of the space above the water in the standpipe, but shall be so designed and constructed as to prevent the ingress and egress of birds, bats or similar animals.

The roof shall be either self-supporting or if the diameter and pitch require, shall be supported by suitable angle and channel rafters or by other satisfactory means and shall be raised above the top of the

cylindrical portion of the tank a sufficient amount to allow a painter's trolley to operate. The roof plates shall have a thickness of not less than $\frac{3}{16}$-inch.

There shall be an ornamental finial on the top of the roof sufficiently strong to support a revolving ladder and a painter's hoist.

A trap door about 2 feet square shall be provided in the roof plate conveniently arranged to be operated from the ladder.

Ladder

SECTION 3.8. On standpipes built without a roof, a steel ladder shall be provided extending from a point 12 feet above the top of the concrete foundation to 3 feet above the top of the shell. The side members of the ladder shall be bent over and fastened to the standpipe shell in such manner as to form suitable hand rails. On standpipes having a steel roof, there shall be provided a fixed ladder extending from a point 12 feet above the top of the concrete foundation to the eave of the roof, and from the eave to the peak of the roof a revolving ladder so constructed and fastened to the finial in the peak of the roof and running on rollers as to permit the ladder being swung around the standpipe. It shall have provision at the lower end for attaching a boatswain's chair from which all parts of the standpipe shell may be reached. On roofs having no greater slope than one to eight, the revolving ladder may be omitted. The side members of the ladder shall be not less than $\frac{3}{8}$-inch by 2 inches, spaced 15 inches apart, the rungs not less than $\frac{5}{8}$-inch, spaced 12 inches apart, and the brackets not less than $\frac{1}{2}$-inch by 3 inches, spaced not to exceed 12 feet apart.

Note: Vertical ladders on high standpipes may be provided with safety cages consisting of 30-inch diameter, 2-inch by $\frac{3}{16}$-inch bar hoops attached to the sides of the ladder at not to exceed 5 foot intervals. Hoops should be reinforced with three 2-inch by $\frac{3}{16}$-inch vertical bars spaced at equal distances around the circumference.

Materials

SECTION 3.9. Steel for plates, rivets and rolled shapes shall conform to the "Standard Specifications for Structural Steel for Bridges" of the American Society for Testing Materials, Serial Designation A-7, Structural Steel, of the latest revision.

All steel to be used for plates for the roof shall be of copper bearing steel or other material resistant to air corrosion. Copper bearing steel shall contain not less than 0.2 per cent copper. In other respects, it shall conform to the above Standard Specifications.

Requirements for design

SECTION 3.10. The loads to be used for designing the standpipe and foundations shall be as follows:

Dead load. The dead load shall be the estimated weight of all permanent construction and fittings.

Live load. The live load shall be the weight of the liquid contents of the standpipe when full. If the roof has a slope of less than 25 degrees with the horizontal, the live load shall also include a snow load of 25 pounds per square foot on the horizontal projection of the roof for all tanks built in locations where snowfall occurs.

Wind load. Wind pressure shall be assumed to be 30 pounds per square foot on a vertical plane surface. In calculating the wind load on the cylindrical surface, six-tenths (0.6) of the above pressure shall be applied to the total area of the vertical projection, and the point of application of the load shall be at the center of gravity of the projected area. Stresses due to wind may be neglected if they are less than 25 per cent of the dead and live load.

In locations exposed to unusual winds, the assumed wind pressure should be taken at 40 pounds per square foot.

Allowable stresses

SECTION 3.11. All parts of the structure shall be designed to resist safely the maximum stresses that can be provided by any combination of the above loads. The maximum stresses in pounds per square inch produced by the above loads shall not exceed the following:

	Pounds per square inch—net section
Tension:	
Tension in cylindrical standpipe plates	15,000
Shearing:	
Shear on rivets	11,250
Bearing:	
Bearing on rivets	22,500
Compression:	
Compression in steel	18,000

For compression members, the permissible unit stress of 18,000 pounds per square inch shall be reduced by the formula:

$$p = \frac{18{,}000}{1 + \frac{L^2}{18{,}000\ r^2}}$$

where p = permissible working stress in compression in pounds per square inch; L = length of member from center to center of connections in inches; r = least radius of gyration of section in inches.

The ratio L/r shall not exceed 175 for roof supporting members.

Basis of stress computations

SECTION 3.12. The pressure possible at the bottom of each course in the standpipe shall be assumed as constant throughout such course, and the tensile stress computed accordingly.

Net sections

SECTION 3.13. Net sections shall be used in calculating the tensile strength in plates and members. In deducting rivet holes, their diameter shall be taken $\frac{1}{8}$-inch larger than the undriven rivets for punched holes and $\frac{1}{16}$-inch larger for reamed or drilled holes.

Minimum thickness

SECTION 3.14. The minimum thickness of metal in the standpipe shell shall be $\frac{1}{4}$-inch for standpipes up to and including 500,000 gallons capacity. For larger sizes $\frac{5}{16}$-inch shall be minimum thickness.

The minimum thickness of metal in the bottom plate shall be $\frac{1}{4}$-inch and the maximum $\frac{1}{2}$-inch. Thickness within these limits shall be determined from the formula:

$$T = \frac{H + 125}{600}$$

in which T = thickness in inches and H = height of standpipe in feet. Thickness used shall be taken as nearest $\frac{1}{16}$-inch above result from the formula.

Rivets

SECTION 3.15. The diameter of rivets shall be not less than the thickness of the thickest plate connected in any joint. The minimum permissible diameter of rivets shall be $\frac{5}{8}$-inch in standpipe plates.

The rivets used shall be of the proper length and diameter to completely fill the rivet holes. Rivets $\frac{5}{8}$-inch and over shall be hot driven and shall have a strong and satisfactory hemispherical or rounded conical head central on the shank and concentric with the rivet holes. Rivets shall have diameters within $\frac{1}{16}$-inch of the diameter of the rivet holes in which they are to be placed. Rivets $\frac{1}{2}$-inch and under may be driven cold. When so driven, the heads may be practically flat.

Attention is called to the requirements that copper bearing steel rivets shall be used in the roof and balcony.

Spacing rivets

SECTION 3.16. The minimum rivet pitch shall not be less than three times the nominal diameter of the rivet.

The maximum pitch along a calked edge shall not exceed two and one-half times the thickness of the thinnest plate connected for single rivet joints or three times the thickness of the thinnest plate in joints having more than one row of rivets, plus, in each case, the diameter of the rivet plus 1.5 inches.

Maximum pitch along uncalked edges shall not exceed thirty times the thickness of the thinnest plate connected.

Edge distance

SECTION 3.17. The distance between the center of the outer row of rivets and the edge of plate shall not be less than one and one-half times the diameter of the rivets. Where edges are beveled, the distance from the center of rivets to the toe of the bevel shall not exceed one and three-fourths times the diameter of the rivets.

Workmanship

Shearing

SECTION 3.18. All shearing shall be neatly and accurately done. All cuts shall be clean without drawn or ragged edges and without splitting away from the sheared edge. Calked edges may be bevel sheared on plates $\frac{5}{8}$-inch thick and under. Plates over $\frac{5}{8}$-inch thick shall have calking edges planned or flame cut by a mechanically guided device.

Beveling

SECTION 3.19. The edges of all plates and butt straps which are to be calked, shall be beveled to an angle of approximately 70 degrees with the plane of the plate for plates up to $\frac{1}{2}$-inch in thickness and for heavier plates the angle shall be increased to approximately 80 degrees.

Punching, reaming and drilling

SECTION 3.20. Rivet holes in material $\frac{1}{2}$-inch thick and under may be punched or drilled full size.

Rivet holes in materials over $\frac{1}{2}$-inch to and including $\frac{3}{4}$-inch thick and in thinner plates for butt joints shall be either drilled from the solid or punched $\frac{1}{8}$-inch smaller in diameter than the nominal diameter of the rivet and then reamed to size.

Rivet holes in material over $\frac{3}{4}$-inch thick shall be drilled.

For the butt joints, rivet holes shall be laid out the required distance from the machined edges of the plates, and where the edges are beveled, the distances shall be laid out from the edge of the surface of the plate with which the rivet heads will be in contact.

The edges of the rivet holes which will be in contact with the rivet heads shall be beveled $\frac{1}{16}$-inch with a twist drill or a rose reamer.

Punched rivet holes shall be made with a punch whose diameter shall not exceed the diameter of the rivet more than $\frac{1}{16}$-inch. The diameter of reamed and drilled holes shall not be more than $\frac{1}{16}$-inch larger than the rivet.

All punched and reamed holes shall be clean cut without torn or ragged edges. Rivet holes shall be accurately spaced. Any notice-

able burrs shall be removed by a tool countersinking not more than $\frac{1}{16}$-inch.

Shaping plates and angles

SECTION 3.21. The plates, straps, angles and other structural shapes shall be accurately shaped to the forms required. All plates, butt straps and structural shapes shall be accurately cold rolled or pressed to the radius of the course for which they are intended. The curvature of any plate, angle or other structural shape shall be uniform throughout, without flat places at the ends.

Scarfing

SECTION 3.22. Where three thicknesses of plate overlap the corner or end of the interior plate shall be scarfed to a thin edge. The beveled surface shall be smooth.

Plates to be scarfed may be heated to a cherry red color, but not hot enough to ignite a piece of dry hard wood when applied to it. If scarfed by pressure such scarfing may be done cold. Most careful attention shall be paid to all scarfing.

Protection of steel

SECTION 3.23. All structural steel received on the work shall be immediately placed upon substantial shores or blocking of sufficient size and strength to prevent any metal from touching the ground, and in such a manner that water cannot collect thereon and so that the material will be protected against bending under its own or superimposed loads. Any steel members which may become bent shall be replaced by new material. Before assembling, the surfaces to be in contact with each other shall be thoroughly cleaned.

Assembling and erecting

SECTION 3.24. All burrs, projections, dirt and rust shall be removed from faying surfaces before the plates are assembled, and the plates shall be securely bolted at the joints in advance of the riveting so as to be in close contact before the rivets are driven.

All the rivet holes in the plates shall coincide, and any holes which do not do so when plates are assembled shall be reamed, and larger rivets shall be used where necessary. The use of drift pins will be allowed only for bringing together the several parts of the structure, and force shall not be used to fair the rivet holes under any circumstances.

If in the opinion of the engineer, the holes in any plate cannot be made to coincide without weakening the joint or producing unsatisfactory work, the contractor shall provide a new and satisfactory plate.

The use of sledges in erection shall be limited to the necessary laying up or forging to close the shell plates around scarfed corners or

butt straps. Care shall be taken to prevent material from falling, or from being in any way subjected to heavy shocks.

Placing bottom of standpipe on foundation

SECTION 3.25. The bottom shall be supported and lowered upon the foundation furnished by the purchaser in such a manner as to prevent sag, in so far as practicable.

The standpipe shall be supported upon a cushion of sand and cement approximately 2 inches in thickness, the materials for which will be furnished by the purchaser at the site of the work. Contractor shall thoroughly mix the sand and cement in equal parts dry and shall spread and carefully level it in such a manner as to provide a uniform bearing over the entire bottom of the standpipe. The bottom shall be absolutely level.

After the standpipe is completed, the sand and cement shall be raked out for a depth of at least 6 inches all around the circumference of the standpipe and around the hole in the foundation for the inlet pipe and be replaced by grout or mortar composed of equal parts of Portland cement and clean, sharp sand, thoroughly filling the raked space and finishing neatly with a bevel on the outside.

Riveting

SECTION 3.26. The field formed head of rivets shall be on the same side of the standpipe as that on which the calking will be done. Heads shall present a uniform appearance.

The rivets shall be driven so as to pinch the plates firmly and all loose or defective rivets with badly shaped heads or with heads out of position, shall be cut out and replaced in a satisfactory manner.

The rivets shall be driven by pressure tools wherever possible. Pneumatic hammers shall be used in preference to hand driving.

Calking

SECTION 3.27. Standpipe shall be made tight by calking. No foreign materials shall be placed between the surface of overlaps.

Calking shall be done with a round nose tool and only by experienced and skilled men.

Testing

SECTION 3.28. After the structure has been erected (but prior to painting) and all seams have been calked, it shall be filled with water furnished by the owner and shall be tested for water-tightness. All leaky seams shall be calked by the contractor, to make the tank watertight. Loose rivets shall be cut out and replaced by new ones but minute leaks at tight rivets may be calked. Any defects revealed by this test shall be made good by the contractor in a satisfactory manner. Tests for water-tightness shall be repeated until the standpipe is perfectly tight. The contractor shall guarantee the tightness of the standpipe.

PART 4. PAINTING

Cleaning and painting

SECTION 4.1. All steel shall be cleaned and painted in accordance with the Specifications for Painting Standpipes as given in the report of the Committee on Painting Standpipes as adopted by the New England Water Works Association at its 1930 Annual Convention and published in the December, 1930, issue of the Journal of the New England Water Works Association.

Note: The purchaser shall state which of the methods of cleaning covered by this specification he desires and which of the paints he wishes applied. (See "Note 1" in the above mentioned specifications.) It is believed that the most effective method of cleaning is by sandblast and that the most satisfactory paint for the interior of the elevated tank or standpipe is the ready mixed red lead paint, as therein specified.

Under certain conditions shop cleaning and the application of one shop coat followed by two field coats of red lead paint may be preferable to field cleaning and painting.

Aluminum paint made by mixing two pounds of aluminum powder with one gallon of aluminum paint vehicle has also demonstrated its efficiency as a protection to steel. Two field coats over a red lead shop coat should be used.

Aluminum powder shall conform to the tentative specifications for aluminum paint (aluminum bronze powder) of the American Society for Testing Materials, Serial Designation D266–28T.

Aluminum paint vehicle shall consist of a varnish containing not less than 50 per cent by weight of non-volatile oil and gum. The varnish shall pass a 60 per cent Kauri reduction test as specified in the Federal Board Standard Specification No. 18 and shall be of such consistency that when thoroughly mixed with albron in the proportion of 2 pounds per gallon of liquid, the paint shall show satisfactory spreading qualities and shall not run or sag when applied to a vertical surface. The paint shall set to touch in not less than 2 nor more than 8 hours and dry hard and tough in not more than 24 hours.

The aluminum powder and vehicle shall be shipped in separate containers and shall be mixed not more than 24 hours before application.

LIST OF SECTION PAPERS

SOUTHWEST SECTION

Houston, Texas, October, 14–17, 1935

MONDAY, OCTOBER 14

Municipal Engineering and Its Relation to Public Water Supplies
E. W. Steel

Finance and Accounting Division

Utility Accounting Methods.....Raymond E. Lee
Water Department Budgets.....T. G. Banks
The Creation and Maintenance of Consumer Good Will.....W. J. Miles

Plant Management and Operation Division

Modern Equipment as an Aid to Efficient Plant Operation
David Morey, Jr.
Pumping Plant Records.....Geo. J. Rohan
The Relation of Spacing to Production From Deep Wells..Sam L. Olson
Water Requirements for Railroad Service.....J. H. Davidson

Distribution and Metering Division

Specifications for Main Construction.....Albert R. Davis
Planning Additions to the Distribution System.....F. M. Veatch
Meter Shop Practice and Meter Maintenance.....M. B. Cunningham
Selection of Proper Size Service Connection and Meter...Lewis A. Quigley

Sanitation and Purification Division

Methods of Correction of Faulty Plumbing.....H. D. McAfee
Proper Sanitary Control of a Deep Well Supply.....Ben L. Grimes, Jr.
Protection of Watershed Areas for Surface Supplies....W. F. Anderson
Cross Connections.....Clyde R. Harvill

TUESDAY, OCTOBER 15

General Session

Certification Standards for the Large and Small Water System
V. M. Ehlers
Obsolescence in Waterworks Equipment and Operating Methods
Wm. W. Brush
Standards of the National Board of Fire Underwriters..Clarence Goldsmith

Superintendents' Round Table

What Cleaning of Water Mains at Evansville, Ind., Has Done (Illustrated)
Charles Streithof
Materials for Service Connections.....Frank M. Murphy

Methods for Handling Emergency Main Repairs.....Henry E. Nunn
Methods and Costs of Street Repairs.....Hubert Davis
Training Schools for Plant Operators and Repair Crews....L. C. Billings
Valve Inspection and Maintenance
Maintenance of Motor Equipment
Relief Labor for Water Works Improvements
Hydrant and Valve Standardization
Emergency Crews for Fire Service

WEDNESDAY, OCTOBER 16

General Session

Ground Water Resources of Texas.....A. H. Dunlap
Plumbing and Its Relation to a Safe Water Supply.....Chester Cohen
Water Board Management of a Water System.....Dr. S. R. Spencer

Address—The Public Works Program and the Water Works
Julian Montgomery

Finance and Accounting Division

Factors to be Considered in Rate Making.....Marvin C. Nichols
Monthly vs. Quarterly Billing of Accounts.....C. B. Coulter

Plant Management and Operation Division

The Value of Efficiency in the Purchase of Pumping Equipment
A. M. Brennecke
Advantages of Making Water Works Structures and Grounds Attractive
Thos. L. Amiss

Distribution and Metering Division

Location Records for Mains and Services.....William K. Van Zandt
Methods for the Prevention of Tampering with Meters...E. C. Johnston

Sanitation and Purification Division

Chlorine-Ammonia Treatment at Houston.....Leonard P. White

ABSTRACTS OF WATER WORKS LITERATURE

FRANK HANNAN

Key: American Journal of Public Health, **12**: 1, 16, January, 1922. The figure 12 refers to the volume, 1 to the number of the issue, and 16 to the page of the journal.

A Note on the System of Public Water Supply in Bermuda. Med. Officer, 1934, **52**: 193–5, 1 fig. From Bulletin of Hygiene, **10**: 3, 176, March 1935. The Watlington waterworks in Bermuda is a supplementary supply taken from a huge fresh water lens, lying upon sea water, and formed by rain falling upon pervious hilly ground. Following the terrain, water flows down the hill sides causing naturally a permanent, though variable, water table. Fresh water is obtained from collecting galleries located at suitable elevations. With pumps it is lifted to a dosing chamber, where it receives proper quantities of lime and sodium aluminate after which it is settled, filtered, and chlorinated.—*Arthur P. Miller.*

The Law as to Water Charges. LEO T. PARKER. Water Works Eng., **87**: 25, 1374, December 12, 1934. Irrespective of business conditions, water company is entitled to rate of return sufficient not only for current repairs, but also for making good depreciation and for replacing the property when it comes to the end of its life, as well as for making a return on the investment. Author cites case of New Haven Water Company, which served a number of communities outside of New Haven and was allowed to charge the same rates in all of them. Courts have ruled that municipally owned plant may charge rates that will make the town a profit. If a city has a valid state law, it may set its own water rates: failing which, the public Service Commission is the rate making body. A large industrial concern which purchases water through one meter and distributes it to consumers is entitled to the reduced rate for large consumers.—*Lewis V. Carpenter.*

Sanitation of Watersheds. A. D. WESTON. Water Works Eng., **87**: 25, 1370, December 12, 1934. All ponds or lakes in Massachusetts having a natural area of 10 or more acres are open to the public for fishing purposes. Of the 1302 ponds, 196 are given over to public water supplies. State laws protect the watersheds and authorize the Department of Public Health to adopt rules and regulations for purpose of preventing pollution. Writer believes that bathing, boating, fishing, and other forms of recreation should be prohibited in sources of water supply. Massachusetts has an extremely low typhoid rate, due in part to vigilance that is maintained in keeping this record. To permit pollution of a water supply, relying on subsequent chemical treatment to render water safe, can certainly not be construed as being in best interests of community served.—*Lewis V. Carpenter.*

Validity of Water Contracts. LEO T. PARKER. Water Works Eng., **88**: 1, 18, January 9, 1935. Water company which enters into contract to sell water to outside sources and later needs the water for its own purposes is required to fulfill its outside contract. It has been held that a city cannot enter into a valid contract to give water away for any purpose. A contract is, however, valid, by which city agrees to pay a water company each year for water an amount identical with tax assessments of water company's property. The courts have held that water customers are not required to lay, or pay for, service pipes, where franchise ordinance fixes the water rates to be charged; mere rules and regulations of water company making such requirement have been declared unreasonable and unjust. It is a well settled law that, although water company may cut off its services for failure on part of the consumer to pay charges, yet Court will grant an injunction requiring water service when the bill is reasonably disputed.—*Lewis V. Carpenter.*

Safeguarding Honolulu Against a Water Famine. FREDERICK OHRT. Water Works Eng., **88**: 2, 66, January 23, 1935. Description of the methods used in reducing per capita consumption of water from 224 to 137 gallons; principally by metering and by increasing water rates. Supply comes mostly from artesian wells.—*Lewis V. Carpenter.*

Producing Colorless Water for Paper Making. J. G. PATRICK. Water Works Eng., **88**: 2, 70, January 23, 1935. For each 100 pounds of finished paper, approximately 1,000 gallons of water is used which must be absolutely free from color. Alum coagulation is frequently unsuccessful in removing organic coloring matter. Activated carbon successfully removes these traces of color and brings out the sparkle so indicative of a good filter effluent. Carbon particles are either retained in settling basin, or caught on sand in filter beds. At Virginia mill, when turbidity of water is low, carbon is added at the pump to remove color. Carbon helps to settle out the small amounts of iron and aluminum naturally present. When water is turbid, carbon is helpful in decreasing the amount of alum required for satisfactory floc formation. At the other plant, carbon tends to increase filter runs.—*Lewis V. Carpenter.*

Water Hammer in Water Supply Engineering. S. LOGAN KERR. Water Works Eng., **88**: 3, 119, February 6, 1935. Velocity of flow in the ordinary distribution system is low and flow is controlled with gate valves. Application of automatic valves for emergency service has frequently resulted in broken mains, due to too rapid shutting off of flow. Many cases of damage are recorded where power supply to centrifugal pumps had failed. Water column in the discharge pipe suddenly reverses and slams the swing-type check valve upon its seat. The automatic check valve causes surges at times. Pressure developed will be of the order of 50 pounds per square inch for each foot per second of velocity.—*Lewis V. Carpenter.*

What the Depression Did to Detroit's Water Finances. C. W. WRIGHT. Eng. News-Rec., **112**: 712–4, May 31, 1934. Large expenditures on water works expansion caused debt of water department to double within decade and

depression, with attendant decreased consumption revenue, inability to collect main assessments, bank failures, and delinquent accounts created serious situation. At height of depression, accumulated delinquency was $1,150,000. Through reduction in operating costs, adoption of judicious collection policy, gradual industrial recovery, and participation in city's bond-refunding operations, department is well on road back to normal conditions. Present valuation of water plant is $116,000,000, against which there is outstanding long-term debt of $63,000,000.—*R. E. Thompson.*

Great Loss to Water Department from Faulty Meter Reading. Eng. News-Rec., **112**: 408, March 29, 1934. As result of long period of faulty reading of meters, Bureau of Water of Cicero, Illinois, had to charge off books in 1933 $164,900, value of water used by consumers in excess of amount billed. Population, in 1930, was 66,602. It had been practice for meter readers to use books containing all previous readings and as result registrations were merely estimated by readers. Nearly 4,000 of the 13,000 meters were found to be out of order, some having been in this condition several years. Many consumers had been billed at minimum rate although great deal more than minimum amount of water had been used. Previous readings are now being withheld from readers and faulty meters are being replaced.—*R. E. Thompson.*

Special Inspection Car Built for Use in Hetch Hetchy Tunnel. Eng. News Rec., **112**: 406, March 29, 1934. Brief illustrated description of storage battery-driven inspection car, built for San Francisco water department to facilitate inspection of 28.5-mile Coast Range tunnel of Hetch Hetchy system. Truck is designed to carry 12 men and 500 pounds of equipment or to haul 4 tons of supplies.—*R. E. Thompson.*

Ten-Year Period of Omaha Water Statistics. THEODORE A. LEISEN. Eng. News-Rec., **112**: 473, April 12, 1934. Brief comparative data are given regarding operation of Omaha water works in 1923 and 1933. In 1923, 3.32 tons of coal were required to pump one million gallons of water; in 1930, last year in which all water was pumped by coal, 23 per cent less. Total cost of all pumpage in 1933 was $260,252, reduction since 1923 of 23 per cent. Consumption increased from 25.4 in 1923 to 29.0 m.g.d. in 1930 and decreased to 26.2 in 1933. Daily per capita consumption in 1933 was 121 gallons. Practically 100 per cent of services are metered and 80 per cent of consumption is accounted for by meter registrations. Bonds outstanding in 1923 ($6,892,000) have been reduced 25 per cent, but additional $2,800,000 has been added in assets from earnings during 10-year period. Average cost per 1000 gallons of water sold in 1933 was 3.39¢ for pumping, 1.08¢ for distribution, 1.97¢ for general expense, 3.07¢ for bond interest, and 6.36¢ for reserves for sinking fund, depreciation, and doubtful accounts, a total of 15.87¢. Income from all sources per 1000 gallons sold was 15.91¢.—*R. E. Thompson.*

Special Master Sets Valuation for Indianapolis Water Company. Eng. News-Rec., **112**: 677, May 24, 1934. Dismissal of federal court suit of Indianapolis Water Company for injunction to prevent enforcement of present rate

schedule was recommended on May 18 by ALBERT WARD, special master in chancery. Recommendations placed value of $20,282,143 on company's property. Company had presented evidence to substantiate its claim to valuation of $28,000,000 and city had submitted evidence to support $14,000,000 valuation. Valuation recommended is $2,000,000 less than that set by state public service commission. Mr. WARD also concluded that 6 per cent return on fair value would not be confiscatory, and stated that return based upon present rates is 6.38 per cent.—*R. E. Thompson.*

Débris Flow from Canyons in Los Angeles County Flood. Eng. News-Rec., 112: 439–40, April 5, 1934. Brief data given regarding mud flows in foothill areas of Southern California which resulted from 30-hour sustained rainfall ending January 1, 1934, sections of towns of La Crescenta and Montrose being severely damaged by moving masses of detritus that debouched from several canyons located in foothills to northeast. From 9 to 17 inches of rainfall was recorded on gages located along foothills and in mountains of area. Intensity was not abnormally high. Entire watershed above towns had been denuded by fire in November and soaking rain had occurred in mid-December.—*R. E. Thompson.*

Wisconsin Sanitation Stimulated Through Federal Activity. Eng. News-Rec., 112: 477, April 12, 1934. Data given from annual report for 1933 of L. F. WARRICK, state sanitary engineer. Eighty sanitation projects, totaling $16,000,000 are entered in PWA program. Ten water works, 17 sewage projects, and 1 combination project make up the $10,900,000 approved to date. There are now 307 public water supplies in state, serving 1,767,000 people, classified as follows: 32 from surface sources, serving 957,000 population; 67 from shallow wells and springs, serving 218,000; 17 from both shallow and deep wells, serving 73,000; and 190 from deep wells, serving 519,000. Sixteen of surface supplies are equipped with purification plants, only 1 being used without treatment. Cross-connection elimination continues. Total of 356 have been reported in 137 municipalities. Connections to non-potable supplies total 190, of which 88 have been given permits, 91 have been eliminated, and 11 are still unsatisfactory. Of 166 with potable supplies, 124 are now reported under permit, 27 have been eliminated, 4 have permission pending, 5 are unsatisfactory, and 6 have inspection pending. Typhoid death rate was 0.56 per 100,000 of population, lowest in history of state. Not one fatal case was traced to water supply.—*R. E. Thompson.*

Defective Elevated-Tank Design Revealed by Earthquake. JOHN HUBER. Eng. News-Rec., 112: 496–8, April 19, 1934. Of hundreds of elevated steel water tanks in territory affected by Southern California earthquake of March 10, 1933, only one, located in South Gate, about 10 miles from Long Beach, the center of disturbance, crashed to complete destruction. Tank, of hemispherical-bottom type, was of 150,000-gallon capacity, 26 feet in diameter, with cylindrical shell 29 feet high. Height of structure to lowest opening was 100 feet. At least 90 per cent of tanks in affected area were built in accordance with National Board of Fire Underwriters' regulations, value of which is

demonstrated by splendid manner in which these structures withstood the severe shocks. Failure of South Gate tank is described and discussed and cause of failure analyzed. Failure was due to lack of balance in material used and also to weakness of details and connections. Effect on other tanks in affected area and revised specifications of National Board of Fire Underwriters are outlined.—*R. E. Thompson.*

Determining Bearing Power in Earth Foundations. H. J. SUMMERS. Eng. News-Rec., **112**: 499–501, April 19, 1934. Details of test conducted at site of proposed elevated steel tank in Torrance, California, are described and results theoretically discussed. It seems entirely within realms of possibility that laboratory tests could be carried out on core samples of soil under consideration, eliminating necessity of loading-platform tests.—*R. E. Thompson.*

New Velocity-Measuring Device Developed in Waterways Laboratory. Eng. News-Rec., **112**: 508–9, Agril 19, 1934. Very sensitive instrument for measuring low velocities in flowing water has been developed in United States Waterways Experiment Station, Vicksburg, Mississippi, by CARL E. BENTZEL. Instrument, which is described briefly and illustrated, is designed to measure velocities of 0.1 to 5 feet per second. It has entirely superseded the Pitot tube and other velocity meters at the station.—*R. E. Thompson.*

Pollution, Turbidity, and Hardness in Mid-West Waters. N. T. VEATCH, Jr. Eng. News-Rec., **112**: 707–8, 1934. Brief general discussion of water supplies of Middle Western States. About 20 per cent of systems, which serve approximately 70 per cent of the population, utilize surface sources. Due to "flashy" character of the streams, surface supplies are subject to wide variations in quality, involving special treatment problems. Ground water supplies are relatively hard and highly mineralized.—*R. E. Thompson* (*Courtesy Chem. Abst.*).

Mulholland Dam Backed by Earthfill Against Downstream Face. Eng. News-Rec., **112**: 558–60, May 3, 1934. Owing to public opinion, which protested against maintenance of 7,437 acre-feet of water in reservoir behind Mulholland Dam, located in Weid Canyon, above and in plain view of Hollywood, 300,000 cubic yards of earthfill has been placed on and against downstream face by Los Angeles Bureau of Water and slope will be planted with trees and shrubs, practically concealing dam from city. Dam, which was constructed in 1924–5, is concrete gravity structure, curved in plan, 200 feet high above foundations, with crest 933 feet long by 16 feet wide, extended by corbels to provide roadway width of 20 feet. Watershed area is small and reservoir is supplied with water piped from other sources. Findings of several boards of engineers and geologists who examined dam are preponderantly for safety of structure and are summarized. Cost of work was about $250,000.—*R. E. Thompson.*

Designing Tunnel Ventilation for Construction Use. G. F. MAUGHMER. Eng. News-Rec., **112**: 597–602, May 10, 1934. Detailed presentation of rapid

and satisfactory method of determining cost and characteristics of tunnel ventilation system. Method was successfully followed in designing plant for some 193,000 feet of tunnels and adits of Colorado River aqueduct project.—*R. E. Thompson.*

Another Great Lakes City Plans a Filter Plant. L. R. Howson. Eng. News-Rec., **112**: 709–12, 1934. Detailed description of 200-m.g.d., filter plant under construction in Milwaukee to treat Lake Michigan water, which has been source of supply for 60 years. Exhaustive investigation indicated that occurrence of gastroenteritis in the city was closely related to the water supply. Plant will be located on made land extending from 700 to 1000 feet into the lake and will consist of four 50-m.g.d. units, each comprising mixing and reaction chamber of flocculator type, providing 60 minutes detention, two-storey coagulation basin, providing retention period of 4 hours, and 8 filters of 6.25-m.g.d. capacity each. There will be 3 clear wells of 30 million gallons total capacity. Wash water storage is provided in a 750,000-gallon reservoir located on adjoining high ground and filters are designed for 36-in. vertical rise of wash water. Provision has been made for continuous sludge removal from lower compartment of coagulation basin in case softening is considered advisable.—*R. E. Thompson* (*Courtesy Chem. Abst.*).

Well Capacity Tested by Measuring Rate Water Can Be Introduced. John A. Oakey. Eng. News-Rec., **112**: 629, May 17, 1934. Method of well testing suggested by L. H. Kessler, consisting of introducing measured supply of water and observing build-up in well, is outlined. This "flow-in" method, which is based on assumption that resistance of substrata is same regardless of direction of flow, is rapid and inexpensive and can be carried out without interfering with drilling operations or waiting for installation of pumping equipment. Ample water supply must be available, of course, and normal water level in well must be such that necessary build-up can occur. Curves are included showing results of testing 2 wells in usual manner and by "flow-in" method.—*R. E. Thompson.*

New York Supply Requires Diversity of Treatment. Frank E. Hale. Eng. News-Rec., **112**: 703–6, 1934. Supplies varying according to source from soft, colorless water of the Catskill Mountains to hard, saline waters from Long and Staten Islands require a wide range of treatment methods: all are naturally corrosive. Infiltration of salt water has in some instances resulted in excessive chloride content of the ground waters. Saline taste develops beyond about 250 p.p.m. and is aggravated by hardness simultaneously present. If sum of chloride and hardness amounts to 400 p.p.m., saline tastes are noticeable even when chloride is less than 250 p.p.m. A zeolite softening plant is in operation at one well and others would be appreciated by the consumers. Iron is a frequent source of trouble, over 0.5 p.p.m. causing rust stains and rendering tea black and bitter. Eight iron removal plants are in operation and others are needed. Mountain supplies, derived from watersheds over 1100 square miles in area, are protected by rigid inspection, long storage (theoretically over 300 days), provision for sewage disposal, and chlorination. Toilets

on all trains are locked while crossing the Croton watershed. At about 110 points, chlorination equipment has been installed. Including city-owned plants, sewage disposal systems on the watersheds, and well stations of private water companies, 52 chlorination plants were in operation in 1933. Chlorine residual of 0.05 to 0.10 p.p.m. is maintained, depending on the location. High residuals are not necessary for elimination of *B. coli*. It has been found on the Catskill system that a chlorine dosage of one-half the chlorine absorption value will eliminate 90 per cent of the *B. coli*; e.g., if absorption value is 0.10 p.p.m., an average dosage of 0.05 p.p.m. will reduce a 10-cc. positive in raw water to zero in the treated. Policy of the city is to chlorinate all surface waters at least twice. In general, *B. coli* is absent in 100-cc. portions of treated water. In 1932, 870 tons of chlorine and 25 tons of bleaching powder were used. Three laboratories are maintained, daily physical and bacteriological examinations being made at 20 points and chemical and microscopical examinations weekly or monthly. Taste-producing microörganisms are not permitted to increase to above 1000 units per cc., and *Synura* not at all. The latter is the most troublesome, imparting cucumber flavor even when present in small numbers. Control of microörganisms is effected by increasing chlorine dosage to as high as 0.9 p.p.m., and by copper sulfate treatment. In 1932, 102 tons of copper sulfate were used. At one plant, 0.6 p.p.m. chlorine effectively destroyed *Achlya* fungus, growth of which had become so heavy that screens in pumping station were blocked. Since 1926, alum and soda ash have been applied at intervals to reduce turbidity in the reservoirs caused by heavy rainstorms. Recently, hydrated lime has been substituted for soda ash as an economy measure. Details of these treatments are included. Typhoid death rate in 1933 was 1.0 per 100,000. There is no reason to believe that filtration will be considered necessary, from a health point of view, in the near future.—*R. E. Thompson* (*Courtesy Chem. Abst.*).

New York Reduces Waste by Stopping Underground Leaks. William W. Brush. Eng. News-Rec., 112: 684–6, May 31, 1934. Prior to 1931, capital improvement work on New York water supply system amounted to about $5,000,000 annually. In 1932, contracts for construction chargeable to capital-outlay account totaled only $87,404, and in 1933 there were no such contracts. During lull in contract construction, comprehensive distribution surveys were conducted, additional leakage surveys were made, new distribution system maps prepared, and new mains laid, latter by gangs from unemployed forces. Total of all leaks located and stopped during past 3 years in Manhattan and the Bronx, where rock fills make possible the existence of large leaks without any surface indication, reached surprising figure of 122 m.g.d. Average costs of first 2 surveys per m.g. saved were $2.22 and $2.36, respectively. During past 3 years, complaint gangs have located and stopped 588 leaks totaling 33.3 m.g.d., average cost per m.g. saved being $5.70. Surveys being undertaken in other 3 boroughs are not expected to yield more than small percentage of Manhattan and Bronx totals.—*R. E. Thompson.*

Discharge of Tainter Gates. Whitney M. Borland. Eng. News-Rec., 112: 846, June 28, 1934. Discussion of Horton's article (Eng. News-Rec. 112: 10, January 4, 1934).—*R. E. Thompson.*

Unconventional Design Features in New Water Softening Plant. Eng. News-Rec., 112: 602–3, May 10, 1934. Water supply of Evansville, Wisconsin derived from deep well, is aggressive, contains 0.7 p.p.m. iron, and has hardness of 165 p.p.m. calcium carbonate and 150 p.p.m. magnesium carbonate (no sulfate). Red water troubles led to construction of softening plant, with ultimate capacity 700,000 g.p.d. It is at present operated at rate of 500,000 g.p.d., being started up each morning and shutting down automatically when 400,000-gallon reservoir is full. Maximum consumption is 350,000 g.p.d. Plant, which is housed in single building 45 feet square, consists of float-switch-controlled deep well pump, delivering to raw water basin occupying almost entire substructure, vertical motor-driven pump which lifts water to Aer-O-Mix aërator, over and under baffled compartment, hydraucone mixer where lime is added, mixing chamber of draft-tube type (paddles operating at 6 r.p.m.), sodium aluminate being added part way down shaft, 2 settling basins, carbon dioxide from oil burning equipment being introduced at outlet, and 2 rapid sand filters. Filter underdrainage system consists of perforated pipes overlain with Wagner filter blocks, which support crushed quartz filter medium. Top size of latter is 0.55 mm. Surface wash system was also provided, consisting of perforated pipes supported 2 inches above sand. Pressure is applied to latter system just prior to commencing wash and is shut off 1 minute before completing wash. Operator can place plant in operation in 20 minutes. Treatment requires 4 pounds hydrated lime and maximum of 0.15 pound sodium aluminate per 1000 gallons. About 3 pounds distillate is consumed per hour in carbonation process. Average total hardness of effluent is 55 p.p.m. Cost of plant was $25,000, one-half of which was for structure.—*R. E. Thompson.*

Composite Roughness in Channels. ROBERT E. HORTON. Eng. News-Rec., 112: 747, June 7, 1934. Reply to W. B. MCMILLAN's discussion (Eng. News-Rec., 112: 268, February 22, 1934) of author's paper (Eng. News-Rec., 111: 652, November 30, 1933) on flow of water in channels.—*R. E. Thompson.*

Rational Rate Structures for Municipal Waterworks. ARTHUR W. CONSOER. Eng. News-Rec., 112: 715–7, May 31, 1934. Establishment of Federal Emergency Public Works Administration has focused attention on profit-making possibilities of municipal water supply installations, development of equitable rate schedules, and recognition of necessity of good accounting procedure. Author believes that investigations made by PWA should lead to development of excellent water-revenue bond law. Theory of two-rate schedule, now generally accepted as most equitable for electric power systems, is discussed and problems in developing rate schedule, including charges for public fire protection, are considered in some detail. Depreciation reserves should take into account long-range planning of additions and improvements.—*R. E. Thompson.*

Dallas, Texas, to make Improvements to Water Supply System. Eng. News-Rev., 112: 679, May 24, 1934. Artesian source of water supply which now serves about one-third of population of Dallas will be gradually replaced with supply from surface sources, change being necessitated by serious decline in

artesian water level. Work will involve expenditure of about $135,000 to provide 24-inch main across Trinity River and million-gallon storage tank. Cost of artesian supply is estimated at 0.0563¢ per 1,000 gallons, as compared with 0.024¢ for filtered surface supply, saving through discontinuing use of wells being estimated at $47,000 per year. In addition, surface supply would be superior in chemical and bacteriological quality. Increased demand for surface water will be supplied from Lake Dallas, reservoir about 25 miles north of city, on fork of Trinity River. Water department plans to spend $800,000 on improvements at reservoir, if additional PWA grant is secured. Latter program would include doubling capacity of present filter plant and rebuilding and replacing mains.—*R. E. Thompson.*

More Water Urgently Needed by New York City. WALTER E. SPEAR. Eng. News-Rec., **112**: 698–702, May 31, 1934. Owing to delay in new water works construction, largely due to legal and financial difficulties, New York has now hardly more than sufficient water to meet present needs. Reasonably adequate rainfall has kept reservoirs full since drought of 1930–1 and concealed impending shortage. On May 4, 1931, Supreme Court confirmed report of special master giving to New York right to develop portion of upper waters of Delaware River within state of New York. Before construction could be started, city's credit became impaired and no funds have since been available for project. In 1930, total water used, including that supplied by private companies and that delivered to other communities, was 1017 m.g.d., 22 m.g.d. (2 percent) less than safe yield of all existing sources, 1039 m.g.d. Had normal annual increase in consumption, averaging 28.2 m.g.d. in previous 10 years, continued after 1930, existing supply would before now have been exhausted. From 1930 to 1932 average consumption decreased, however, to 957.6 m.g.d. In 1933, average was 958.2 and during first 3 months of 1934 was 1014, about 85 m.g.d. greater than in corresponding period of 1933 and 34 m.g.d. greater than previous maximum in 1930. Delaware project as modified by Supreme Court decision of May 4, 1931, contemplates development of two tributaries of Delaware River, the East Branch and the Neversink River, and of upper Rondout Creek, tributary of the Hudson, by construction of reservoirs on those streams and delivery of estimated supply of 540 m.g.d. through pressure-tunnel aqueduct, estimated cost being $272,587,000. Sources which could be quickly developed as emergency supplies would possibly yield 96 m.g.d., making total yield 1135 m.g.d., which must suffice until final stage of Delaware project, at present at standstill, is completed. Estimates indicate that city will have exhausted supply, including that from emergency sources, in 1940 and it is believed hardly possible that water could be obtained from Delaware by that time. Situation is therefore serious. Reasons are given why it is believed that universal metering would not result in curtailment of consumption to same extent in New York as has been the case in other cities. All industrial premises and 28,000 domestic consumers are now metered. Average daily per capita consumption in 1933 was 128 gallons. Metering, it is believed, would reduce this figure by not more than 10 gallons and average daily consumption by not more than 80 m.g. Cost of universal metering is conservatively estimated at $30,000,000. House to house inspections made in past have

indicated that 75 m.g.d. could be saved in this way at annual cost of $200,000; i.e., as much as by metering, and at fraction of cost. Additional saving of 70 m.g.d. as of 1939 might be effected by underground leakage reduction.—*R. E. Thompson.*

General Improvements Made in Many Water Systems. Eng. News-Rec., 112: 686, May 31, 1934. Brief outline of improvements made in water systems chiefly with aid of CWA forces and PWA grants. Buffalo is increasing area served by high-pressure fire protection system from 387 to 900 acres. Universal pipe in 6-foot lengths is being used, owing to greater flexibility under severe limitations of space for construction. In Cleveland, 23-million gallon concrete filtered water reservoir, 326 x 306 feet in plan and 32 feet deep, was built at cost of $386,000. Roof has membrane waterproof coating protected with layer of asphalt. Extensive repairs are also being made to filter tanks at Division Ave. plant. Waterproofing of interior of tanks, after cutting out damaged and inferior concrete, was effected with layers of asphalt and impregnated fabric. Asphalt planking was placed in vertical sides to depth equal to combined thickness of gravel and sand layers, the gravel regraded, and sand thoroughly cleaned. At Minneapolis, pitometer survey is being conducted by contract on guarantee basis. Net revenue accounts for 72 per cent of water pumped and public and free use is estimated at 10 percent, leaving 18 percent unaccounted for. In addition, 2 coagulation basins are under construction. In Chicago, 653,615 fixture leaks were found on 359,148 premises; 35,000 were repaired, stopping waste estimated at 13.4 m.g.d. Total wastage in whole system is estimated at 200 m.g.d. Extensive replacement and reconditioning of valves, hydrants, and meters was also carried out. At St. Louis, a number of changes and additions costing $30,000 were made at Howard Bend filter plant.—*R. E. Thompson.*

Building a Filter Plant Financed by the RFC. Eng. News-Rec., 112: 692–7, May 31, 1934. Detailed, illustrated description of problems encountered in financing and constructing new 6-m.g.d. filter plant for Wilmette, Ill. Supply was formerly obtained from Evanston. Direct-obligation bond issue was defeated by electors in November, 1930, and after favorable vote on issue of water-revenue bonds in December, 1931, it was found impossible to sell them. In October, 1932, agreement was made for sale of securities to RFC; construction was commenced on February 20, 1933 and plant was in operation on March 1, 1934. Site chosen was between a County Club and the lake, and owing to objections of club members, height of building was kept very low, necessitating substantial portion of structure being below lake level, with attendant complications in construction. In addition, wash water and basin drainage could not be discharged into lake, necessitating construction of reservoir discharging into sewer for storage of these waters. Facilities were provided for returning wash water to raw water if this is found beneficial. Intake consists of 33-inch diameter steel pipe, terminating 3000 feet from shore, where water is 23 feet deep, in steel intake drum with ports protected by wooden racks. Maximum port velocity when filters are enlarged to twice present capacity will be 0.10 foot per second. Mixing chambers, of which there are four, are provided

with variable-speed, electrically-operated stirring arms, adjustable as to height and angle of inclination. Mixing basins provide nominal retention period of 35 minutes and sedimentation basins, of 4 hours. There are 4 filters, each of 1.5 m.g.d. nominal capacity; sand bed being 30 inches deep and of 0.55 mm. effective size. Wash water pump will give vertical rise of 42 inches per minute. Filtered water reservoir has capacity of 1.28 million gallons.—*R. E. Thompson.*

Large Derricks Supplement Cableways in Boulder Dam Work. Eng. News-Rec., **112**: 832–3, June 28, 1934. Three steel stiff-leg derricks have been installed to facilitate construction of Boulder Dam. Brief details given, with illustrations.—*R. E. Thompson.*

Mottled Enamel Teeth and the Fluorine Factor. JACK WYATT. Southwest Water Works J., **17**: 2, 9, 1935. Good summary of conditions in the Texas Panhandle, which is the largest known endemic area in the U. S.—*O. M. Smith* (*Courtesy Chem. Abst.*).

Regulation of Public Utilities. W. N. GLADSON. Southwest Water Works J., **17**: 3, 9–11, 1935. Discussion of efforts of State of Arkansas in regulation of public utilities.—*O. M. Smith.*

Operation and Maintenance of Fire Hydrants. T. G. BANKS. Southwest Water Works J., **17**: 3, 14–15, 1935. Keynote to hydrant and valve maintenance is systematic inspection, followed up by prompt repairs; which presupposes adequate records, materials suited to local conditions, and competent, experienced, and trustworthy workmen. Detailed discussion of methods of inspection adhered to in Oklahoma City and of measures taken to keep down costs.—*O. M. Smith.*

Journal of the New England Water Works Association, Vol. 48, No. 4, December 1934. Hydraulic Analysis of Water Distribution Systems by Means of an Electric Network Analyzer. THOS. R. CAMP and H. L. HAZEN. 383–407. Analogy between formulae $h = cQ^z$, for hydraulic friction, and $V = kI^z$, for electric friction, suggested use of electric calculating board, or network analyzer, for solving hydraulic problems. Computation of pressure gradients in water distribution system of Warwick, R. I., for two fire-load conditions is presented to illustrate application of electric network analyzer to solution of practical problems. Equations, conversion factors, tables of constants and resistor assignments, and possible errors are given. While problem solved was in itself simple, network analyzer proved more precise than hydraulic flow formulae which furnish data. In the **discussion**, Professor BABBITT evaluates network analyzer, and while commending able work of CAMP and HAZEN, points out its very definite limitations. **Expansion of the Freeman Method for the Solution of Pipe Flow Problems.** W. E. HOWLAND. 408–411. Brief explanation of author's modification of JOHN R. FREEMAN's method for pipe flow computation which has proven useful, particularly for systems consisting of separate loops not interconnected in many places. Suggested by CAMP's

work on network analyzer. **Public Works for Economic Stabalization.** WILLARD T. CHEVALIER. 412–422. Specific mission or objective of RFC, CCC, CWA, TVA, and PWA justifies their existence as part of program to achieve relief, recovery, and some measure of reform. Public works program has been sorely handicapped by absence of regional plan and definite program. Necessity for long term regional planning is stressed. **A Recent Improvement in the Centrifugal Casting Process.** H. W. STUART and S. E. LINDERMAN. 423–440. Recent discovery that centrifugal pipe could be cast without chill in steel molds has improved quality of pipe tremendously. Operation of casting machine is described in detail. Recent tests have demonstrated that pipe cast by new method is more resistant to handling shocks, due to high strength combined with ductility. **Public Works and Business Stabilization.** R. S. MERIAM. 441–450. Erstwhile barefoot boy from Maine and present professor at Harvard bemoans failure of public works program to hasten business recovery. Government's obviously unfair policy of forcing reduction in already inadequate salaries of railway and insurance presidents, while in position to increase princely stipend of wage earners to unprecedented heights, must inevitably thwart private initiative and lead country to ruin. In masterly refutation of Col. CHEVALIER'S contention that public works are beneficial, noted economist proves that normal recovery of 1932 has been retarded by projects of Brain Trust. **Modern Aspects of Water Purification.** E. SHERMAN CHASE. 451–459. Review of American water purification practice, presented before Institute of Water Engineers at Harrogate, England, in 1931.—*T. F. Donahue.*

The Significance of Ammonia for Chlorine Consumption of Water. M. L. KOSCHKIN und E. M. SPECTOR. Zeitschrift für Hygiene und Infektionskrankheiten, **116**: 6, 688, February 1935. In this fourth communication, influence of the physical factors light and temperature on chlorination of preammoniated water is discussed. Chlorine consumption increases in daylight with increasing light intensity; after preammoniation, values remain lower than without this treatment. Chlorine consumption increases also with temperature, but in preammoniated water to a lesser extent; ability of ammonia and ammonium salts to reduce chlorine consumption seems to fall off as temperature increases. Chlorine fixation at boiling temperature according to procedures of FROBOESE and ELMANOWITSCH-ZALESKI respectively, is a multiple of that at room temperature and is but little influenced by preammoniation, being reduced at most 10 percent, whether hay-extract, or peptone, was used. Fluctuations in chlorine consumption under influence of temperature, or of light, are less after preammoniation; this is of practical importance.—*Mans.*

Investigations on the Reactions between Trass and Electrolyte Solutions and on Their Applicability for Water Softening. A. STEOPOE and G. TIMIS. Buletinul de chimie pura si aplicata als societatii Romane de chimie, 36: 1/6, 45, December 1933. Trass samples from Slanic and Dej by their thermal dehydration in stages, their solubility in cold hydrochloric acid, and their partially exchangeable calcium content, gave evidence of appreciable contents of zeolitic constituents. In contact with lime water, calcium is taken up to an

amount proportional to initial concentration, the amount being increased after preheating to 100° or 400°C., but decreased after heating to 250°; the exchange capacity may be raised by pretreatment with hot hydrochloric acid, or with sodium hydroxide, and washing; but diminished by treatment with alum. Calcium and magnesium are taken up from solutions of their salts in amounts corresponding to content of zeolitic alkali, exchange capacity being raised after heating to 400° for half an hour, but lowered after heating to 200° or to 500°C., or by prolonged heating to any temperature. When the material is treated with 20 percent sodium hydroxide, heating up to 300°C. has no longer any influence. Exchange capacity is increased by treatment with chemicals, if thereby the zeolitic bound alkali is increased, as is the case by treatment with lime water and washing with salt solution, or with sodium hydroxide. Magnesium is taken up to a lesser extent than calcium; cations more easily from sulphate solution than from chloride. Property of extracting manganese and alumina from water is limited; ferric salts hydrolize and, due to neutralization of the acid by the alkali of the trass, are completely precipitated as hydroxides, interfering with base exchange. Tap water was softened by tuff sand, the softening capacity increasing with decreasing size of the sand, proportionally to the square of the radius. Exchange value of the tuff sand was raised only by treatment with 20 percent sodium hydroxide, or by hydrochloric acid followed by treatment with alkali and when exhausted was regenerated with cold 20 percent salt solution.—*Manz.*

Water Softening by Zeolites: Sanitary Regulations Prescribed for Domestic Softeners. H. F. KUISEL. Schweiz. Verein von Gas- und Wasserfachmännern, Monatsbulletin **15**: 2, 37, February 1935. Swiss food regulations for domestic water softeners require chemical and bacteriological purity in water to be softened and also in salt for regeneration; salt of ordinary grade may not be used. Owing to their unpleasant taste (excess sodium and calcium chloride), first rinsings after regeneration are to be rejected; amount of water to be run to waste must be specified. In view of the high sodium bicarbonate content of many well known mineral waters, the alkali content of softened tap water can not be considered as injurious to health. Investigations on various zeolite materials suitable for small apparatus placed them as follows in order of decreasing base exchange capacity: American webbolite > doucil > sodium permutite G > natrolite > neopermutite > zerhyd. After regeneration, pH value of soft water is raised; the effect decreasing at rate which varies according to material under test; no general relations between base exchange capacity and hydrogen ion concentration have been established; each zeolite shows specific behavior. For webbolite and, to a somewhat less extent, for natrolite, pH and soft water output are inversely correlated. After a flow of approximately 40 liters water through 2 kilograms zeolite material, chloride content declines to raw water value. Due to scouring action, small amounts of alumina, or minute particles of zeolite are always noticeable; in softened Zürich tap water up to 0.5 p.p.m. was detected; alumina content of soft water decreases during softening by approximately one half. Water with residual hardness of 30 to 40 p.p.m. is recommended in preference to zero water, to avoid corrosion of boiler and hot water apparatus and objectionable alkaline flat taste.—*Manz.*

The Dechlorination of Water. RUDOLF ADLER. Gesundheits-Ingenieur, 58: 18, 272, May 1935. Highest degree of purifying and sterilizing action by chlorine is attained with doses exceeding chlorine demand of the water, the remaining excess being eliminated by additional treatment. Dechlorination by activated carbon involves periodical revivification, as often as the carbon surface becomes overloaded by deposits of adsorbed matter from the water, or by carbon oxidation products, such as mellogens. Substances such as ammonia, proteins, and humins, forming stable chlorine compounds, cause a carbon bed to break down long before its capacity to remove free or hypochlorite chlorine is exhausted. Bacterial enrichment may also occur to objectionable extent in carbon layers where flow is inactive and chlorine concentration under germicidal level. As a new dechlorinating agent, catarsit, a double compound $m\ CaO \cdot n\ CaSO_3$, consisting essentially of neutral calcium sulfite, is recommended. Calcium sulfite has the low solubility of 57.9 p.p.m. and no noticeable adsorptive power. When water flows through catarsit, sized from 4 to 8 mm. for greater freedom of flow, chlorine combines with the dissolved as well as with the solid sulfite; with catarsit layer of suitable depth, all free chlorine and also chlorine compounds with proteins etc. are removed. Increase in hardness is but small. Rate of dechlorination increases with increasing water temperature. Other things being equal, an initial chlorine content of 5 p.p.m. is entirely removed by 18-inch layer of catarsit, as against approximately 8 feet of carbon. As during flow of water through catarsit, continuous solution of sulfite takes place, both as such and, after oxidation, as the more soluble calcium sulfate, the catarsit surface is continually renewed and complete dechlorination is secured.—*Manz.*

On the Influence of Local Rainfall on the Bacterial and Coli Contents of Drinking Water Samples. H. GROSSMANN. Zeitschrift für Gesundheitstechnik und Städtehygiene, 27: 4, 114, April 1935. Correlation between local precipitation and bacterial findings on a number of small supplies from springs with varying geographical and topographical conditions in a mountainous region is discussed. Sampling by skilled staff being impossible, sterile bottles were distributed with instructions and questionaires covering local conditions, weather during sampling and for a week previous, etc. More accurate data relating to rainfall were secured from records of the Prussian meteorological office stations. Results revealed the fact that apparently good bacterial counts are correlated to low rainfall; out of 62 dry weather samples, with low gelatine counts, only 15 showed positive coli tests, while after heavy rainfall, 53 out of 58 samples, with high counts, were also coli positive. This proves the necessity to take into account meteorological conditions in control of water supplies over long periods, as a series of unobjectionable findings during drought, or freezing, might create a false feeling of security.—*Manz.*

Chemical Behavior of Concrete and Durasbestos. L. W. HAASE. Zeitschrift für Gesundheitstechnik und Städtehygiene, 27: 4, 119, April 1935. Method of manufacture of Durasbestos or Eternit water mains by building up from thin films of asbestos-cement mixture produces increased resistance to internal pressure, but not to bending. Both asbestos cement and concrete pipes give off soluble calcium salts, especially calcium hydroxide, to water; in

asbestos-cement pipe with high rates of flow this alkalization quickly declines and after prolonged use no influence on palatability is noticeable. It is believed that after initial solution has taken place, the filamentous structure of pipe surface and the deposition of calcium carbonate and iron or manganese precipitates between the fibers hinder access of water to deeper layers and therefore diminish rate of corrosion, even by waters of acid character. In general, asbestos-cement pipe is more stable in quickly running than in standing water; in the latter, calcium carbonate solution is accentuated and disintegration of the upper layers takes place. A protective layer of crystalline calcium carbonate, which is formed only at a definite alkalinity, is more resistant; in stagnant layers, between the asbestos filaments, amorphous non-protective calcium carbonate precipitates; that may explain corrosion in dead ends. Difference in behavior between running and standing water may account also for the fact that at low flow sulphates, sulphides, and free hydrogen sulphide are detrimental. Also with harder inaggressive waters it is deemed advisable to apply a bituminous phenol-free lining, unless a sufficient continuous flow can be maintained. In some cases it was observed that the internal surface showed an excessive degree of after-hardening, possibly due to formation of double compounds between calcium carbonate and the magnesium silicate of asbestos.—*Manz.*

Mottled Enamel in Texas. H. Trendley Dean, R. M. Dixon and Chester Cohen. U. S. Public Health Reports, 50: 13, 424, March 29, 1935. This paper presents the results of a study made during November and December 1934. A total of 66 cities, towns, or rural communities in 44 counties was visited and 3,723 school children were examined. Children generally of the fourth, fifth, and sixth grades were examined. The purpose of the survey was to obtain general information relative to the extent of the affected territory and a rough index of the degree of severity of the mottled enamel being produced. The results of the study are summarized as follows: (1) the Panhandle-west Texas region constitutes the largest mottled-enamel area in the United States; (2) of 53 communities surveyed in 37 counties in the Panhandle region, only 6 could be classified as "negative" or "border line"; (3) the fact that the municipal water supplies of such large cities as Amarillo, Lubbock and Plainview contain the causative factor of mottled enamel in sufficient concentration to produce this hypoplasia in a high percentage of their children has developed an acute and urgent public health problem; (4) an endemic area of unknown size is reported in east central Texas between Austin and Dallas where of 13 communities surveyed, only 2 were "border line" and none "negative."—*A. W. Blohm.*

Efficient Regulation of an Air-Conditioning System. Bulletin 4001, 1935. Leeds & Northrup Company, Philadelphia, Pa. This well-illustrated booklet gives useful information relative to electrical thermometers.—*A. W. Blohm.*

Boulder Dam. G. Ross Robertson. Ind. Eng. Chem., 27: 3, 242-9, 1935. As result of extensive research program, low-heat cement is being used in Boulder Dam, additional cooling being accomplished by running cooling water

through one-inch pipe embedded in concrete mass. Block system of pouring is used and final bonding is accomplished with extremely fine grouting cement forced in at high pressure. Specifications for the cement and details of mixing and pouring are given. With 14 illustrations and bibliography.—*Selma Gottlieb.*

Oxygen Removal from Boiler Feed Water by Sodium Sulfite. KENNETH A. KOBE and WARD L. GOODING. Ind. Eng. Chem.; **27**: 3, 331–3, 1935. Reaction goes rapidly to completion at temperatures of boiler, or of hot process softener, but is less rapid at outdoor temperatures. In latter case, reaction is not efficient at higher pH values. Traces of order of 0.1 p.p.m. of copper sulfate are effective as catalysts; but large excess is apparently inhibitory, as are certain organic compounds in water. Process is desirable because of cheapness, speed, and nature of end-products.—*Selma Gottlieb.*

Organic Matter in Boiler Feed-Water Treatment. D. K. FRENCH. Ind. Eng. Chem., **26**: 1319–24, December, 1934. Tannins added to boiler water with regular softening chemicals retard scale formation, produce non-settling sludge, and even remove old scale. Not all organic materials of tannin origin are effective, but chestnut oak extract has been successful. Action of tannins on old scale is both chemical and physical. In acid treatment of carbonate scale, tannins inhibit action of acid on metal. They reduce corrosion by reaction with electrolytes, by formation of iron tannate film, and, probably most effectively, by absorption of oxygen. By decomposing to carbon dioxide and carbonates at high boiler temperatures, tannins prevent formation of sodium hydroxide and therefore prevent caustic embrittlement. Tannins decrease foaming, due to clarifying properties of precipitates formed and to properties of surface film.—*Selma Gottlieb.*

Corrosion Problems. JOHN JOHNSTON. Ind. Eng. Chem., **26**: 1238–44, December, 1934. Thermodynamics provide more widely applicable approach to corrosion problems than does electro-chemical theory. Stability of substance toward corrosion depends not only on inherent properties, but also on environment. Rates of reaction are not proportional to thermodynamic driving forces, but always increase rapidly with rising temperature. In sound metals, thin scale, rapidly formed at first, prevents further diffusion of corroding agent to metal and slows corrosion. Alloying or coating materials effective in producing this effect on iron can be determined only empirically, especially since requirements of film cannot yet be stated. In presence of moisture (atmospheric conditions), hydration and carbonation of film introduce complications. Corrosion tests must correspond to actual conditions of exposure and reagents used must not destroy film. Corrosion under water depends on kind and concentration of ions present, especially hydrogen ion, dissolved gases, local differences of electrical potential, electrical conductance of solution, etc. No single test can reflect behavior of metal except under conditions substantially identical with those of exposure. Initial attack is at some discontinuity; but attack, once begun, provides nuclei for its spread. Underground corrosion depends more on nature of soil than on metal. Pres-

ence of potential difference indicates tendency to corrode, but tells nothing as to rate of reaction, nor as to subsidiary reactions tending to nullify main reaction. Relative position in electrochemical series is not significant, unless account is taken of real ion concentrations at metal surface and of any film (e.g. gaseous or oxide) altering nature of electrode, and increasing its effective nobility. Whenever contact of dissimilar metals is unavoidable, electrolytic effects can be minimized by having baser metal surface large relatively to that of nobler metal; reverse condition causes pitting.—*Selma Gottlieb.*

NEW BOOKS

Parvianalyse chimique et toxicologique des eaux potables. P. MOUNIER. Paris: Editions Médicales N. Maloine. 295 pp. Reviewed in Ann. fals., 27: 361, 1934. From Chem. Abst., 28: 6897, November 10, 1934.—*R. E. Thompson.*

Swimming Bath Water Purification. F. WILKINSON and F. J. FORTY. London: The Contractor's Record, Ltd. 12s. 6d. From Chem. Abst., 28: 6897, November 10, 1934.—*R. E. Thompson.*

Lehrbuch der Grundwasser- und Quellenkunde. Konrad Keilhack. 3rd edition, revised. Berlin: Borntraeger. 575 pp. M. 40. From Chem. Abst. 29: 3081, May 10, 1935.—*R. E. Thompson.*

Corrosion. Causes and Prevention. FRANK N. SPELLER. 2nd edition. New York: McGraw-Hill Book Co., Inc. 687 pp. $7. From Chem. Abst., 29: 3970, June 20, 1935.—*R. E. Thompson.*

Cours de chinie industrielle. Fasc. 1. Eaux, origine, composition, correction, utilisation. Analyse. L. BOURGOIN. Montreal: L'École Polytechnique de Montreal. 518 pp. Reviewed in Chimie & industrie, 33: 624, 1935. From Chem. Abst. 29: 4116, June 20, 1935.—*R. E. Thompson.*

Water Supply and Treatment. CHAS. P. HOOVER. Washington: National Lime Association. 143 pp. From Chem. Abst., 29: 4116, June 20, 1935.—*R. E. Thompson.*

L'eau. M. P. OTTO. Paris: Librairie Hachette. 190 pp. F. 12. Reviewed in Chimie & industrie, 33: 356, 1935. From Chem. Abst., 29: 4116, June 20, 1935.—*R. E. Thompson.*

Das Wasser in der Industrie und im Haushalt. J. LEICK. Dresden: Th. Steinkopff. 118 pp. Bound, about M. 9.20. From Chem. Abst., 29: 1916, March 20, 1935.—*R. E. Thompson.*

Water Sterilization. London: Imperial Chemical Industries, Ltd. 50 pp. Reviewed in Chem. Age, 31: 336, 1934. From Chem. Abst., 29: 1916, March 20, 1935.—*R. E. Thompson.*

Handbuch der Hydrologie. EMIL PRINZ and ROBERT KAMPE. **Bd. II. Quellen. (Süsswasser- und Mineralquellen).** Berlin: J. Springer. 290 pp. M. 24. 50. From Chem. Abst., 29: 2271, April 10, 1935.—*R. E. Thompson.*

Über Fettabscheider in der Grundsütcksentwässerung und ihre Prüfung. CARL REICHLE, FRITZ MEINCK and HEINRICH KISKER. Berlin: C. Heymann. 48 pp. M. 3. From Chem. Absts., 29: 2271, April 10, 1935.—*R. E. Thompson.*

The Pharmacological Action of the Harrogate Drinking Waters. W. BAIN. London: Churchill and Co. Ltd. 5s. From Chem. Abst., 29: 2637, April 20, 1935.—*R. E. Thompson.*

Boiler Feed Water: Its Effects, Treatment, and Analysis. PERCY G. JACKSON. 3rd edition, revised. London: C. Griffin and Co., Ltd. 6s. From Chem. Abst., 29: 2637, April 20, 1935.—*R. E. Thompson.*

Abastecimiento de agua potable. JUAN LAZARO URRA. Madrid: Revista de obras publicas. 502 pp. Ptas. 46. From Chem. Abst., 29: 2637, April 20, 1935.—*R. E. Thompson.*

Dampfkesselschäden, ihre Ursachen, Verhütung und Nützung für die Weiterentwicklung. ERNST PFLEIDERER. Berlin: J. Springer. 259 pp. M. 24. Reviewed in Chimie & industrie, 33: 852, 1935. From Chem. Abst., 29: 4116, June 20, 1935.—*R. E. Thompson.*

Indicators. Distant Indication and Control for Water Undertakings. London, Action Lane Works: Evershed and Vignoles Ltd. 24 pp. From Chem. Abst., 29: 4116, June 20, 1935.—*R. E. Thompson.*

Water Supply in Buildings. A. W. MOSELEY. Evanston, Ill.: Mumm Print Shop. 91 pp. $1.15. From Chem. Abst., 29: 4862, July 20, 1935.—*R. E. Thompson.*

Sedgewick's Principles of Sanitary Science and Public Health. SAMUEL C. PRESCOTT and MURRAY P. HORWOOD. Enlarged edition. New York: The Macmillan Co. 654 pp. $4.25. From Chem. Abst., 29: 4862, July 20, 1935.—*R. E. Thompson.*

Limnology. PAUL S. WELCH. New York: McGraw-Hill Book Co., Inc. 471 pp. $5. From Chem. Abst., 29: 5210, August 10, 1935.—*R. E. Thompson.*

Arch Dam Investigation, Volume II. Cloth; 6 x 9 inches; pp. 542. Published by United Engineering Trustees, Incorporated, for the Engineering Foundation, 29 W. 39th St., New York. Volume I, $5; Volume II, $4; Volume III, $3. Special rates to members of the Founder Societies. Reviewed in Eng. News-Rec., 112: 847, June 28, 1934.—*R. E. Thompson.*

Water Supply Problems and Developments. W. H. MAXWELL. Cloth; 6 x 9 inches; pp. 254. New York: Pitman Publishing Corporation. London: Sir Isaac Pitman and Sons, Ltd. $6. Reviewed in Eng. News-Rec., 113: 280, August 30, 1934.—*R. E. Thompson.*

Civil Engineering Handbook. LEONARD CHURCH URQUHART. Flexible; pp. 885. McGraw-Hill Book Co., Inc. New York and London. $5. Reviewed in Eng. News-Rec., 113: 280, August 30, 1934.—*R. E. Thompson.*

Water Power Engineering. H. K. BARROWS. 2nd. edition. Cloth; 6 x 9 inches; pp. 762. McGraw-Hill Book Co., Inc., New York. $6. Reviewed in Eng. News-Rec., 113: 536, October 25, 1934.—*R. E. Thompson.*

Power Supply Economics. JOEL D. JUSTIN and WILLIAM G. MERVINE. Cloth; 6 x 9 inches: pp. 276. John Wiley and Sons, Inc., New York. $3.50. Reviewed in Eng. News-Rec., 113: 536, October 25, 1934.—*R. E. Thompson.*

Hetch Hetchy. M. M. O'SHAUGHNESSY. Cloth; 9 x 12 inch; pp. 133. Newbegin's Book Shop, San Francisco, Cal. $5. Reviewed in Eng. News-Rec., 113: 536, October 25, 1934.—*R. E. Thompson.*

Applied Hydraulics. HERBERT ADDISON. Cloth; 6 x 9 inches; pp. 409. New York: John Wiley and Sons, Inc. $5.50. Reviewed in Eng. News-Rec., 113: 831, December 27, 1934.—*R. E. Thompson.*

Text Book on Hydraulics. GEORGE E. RUSSELL. 4th edition. Cloth; 6 x 9 inches; pp. 447. New York: Henry Holt and Co. $3.90. Reviewed in Eng. News-Rec., 113: 831, December 27, 1934.—*R. E. Thompson.*

Public Utility Valuation for Purposes of Rate Control. JOHN BAUER and NATHANIEL GOLD. Cloth; 6 x 9 inches; pp. 477. Macmillan Co., New York. $3.50. Reviewed in Eng. News-Rec., 113: 830, December 27, 1934.—*R. E. Thompson.*

Hydraulics. ERNEST W. SCHODER and FRANCIS M. DAWSON. 2nd edition. Cloth; 6 x 9 inches; pp. 429. New York: McGraw-Hill Book Co. $3.50. Reviewed in Eng. News-Rec., 113: 831, December 27, 1934.—*R. E. Thompson.*

Elements of Water Supply Engineering. EARLE LYTTON WATERMAN. Cloth; 6 x 9 inches pp. 302. New York: John Wiley and Sons, Inc. $3.50. Reviewed in Eng. News-Rec., 114: 174, January 31, 1935.—*R. E. Thompson.*

The Flow of Water in Flumes. FRED C. SCOBEY. Technical Bulletin No. 393, United States Department of Agriculture. Paper; 6 x 9 inches; pp. 99. Washington, D. C.: Superintendent of Documents. 15¢. Reviewed in Eng. News-Rec., 114: 174, January 31, 1935.—*R. E. Thompson.*